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Design Guide for Steel Railway Bridges

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FOREWORD

This publication has been prepared to provide a comprehensive yet relatively concise introduction to the design of steel and composite railway bridges, with particular reference to design in accordance with BS 5400 *Design of steel concrete and composite bridges*.

The publication has been prepared by Mr D C Iles (The Steel Construction Institute), based on a number of substantial contributions from practising senior engineers and with the assistance of an Editorial Advisory Group.

The main contributions were made by the following companies:

Atkins Rail
Corus Rail Consultancy
Scott Wilson Railways
Network Rail
Fairfield Mabey Ltd

Guidance in drafting Sections 6, 7 and 8 was provided by Mott MacDonald. The worked example of a U-frame bridge is based on an example provided by Cass Hayward and Partners.

The SCI is very grateful to these companies for their contributions.

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SUMMARY

This guide offers an introduction to the design of steel and composite railway bridges. It presents a review of the particular requirements for carrying railway traffic and interfacing with railway infrastructure and, since construction issues are particularly acute for replacement bridges, describes the constraints and options for construction. Typical forms of construction for short to medium span bridges are described and simplified cross sectional arrangements are illustrated. A key consideration for the selection of railway bridges is a safe and reliable structure that can be constructed and maintained with minimal disruption to railway passengers and traffic – steel railway bridges meet these requirements particularly well.

The guide explains that design of railway bridges in the UK is governed not only by BS 5400 but also by the comprehensive additional requirements of the railway authorities, notably Network Rail. Design procedures are described and detailed aspects of design for strength, for fatigue endurance and for deformation performance are discussed. A range of typical details is illustrated, with comments about the factors that need to be considered when selecting and designing them.

A worked example of a single span half through bridge with a composite deck carrying two tracks is presented. It illustrates the main features in determining the appropriate bridge details and in verifying the adequacy of the bridge.

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1 INTRODUCTION

Steel has been used in the construction of railway bridges since Victorian times, both in major long-span structures and in more modest spans, such as over local highways. Many of these older steel bridges are still in service, thanks to an on-going programme of maintenance, refurbishment and upgrading to suit changing requirements. Today, most railway bridges are built as replacements for earlier structures, although some entirely new structures are being built on new alignments or routes, most notably for the Channel Tunnel Rail Link.

For replacement bridges, steel construction can achieve shallow construction depths, which is essential where the track level is fixed yet sufficient underline clearance is needed for highways or other services below the bridge. Steel construction is suited to prefabrication and preassembly and, because of its relatively modest self weight, can be transported or lifted into position in the very short periods during which the railway can be closed to traffic.

For bridges on new alignments, where there is greater freedom with the vertical profile of the route, greater construction depths can be used and this opens up the possibility of the use of slab-on-beam composite construction, in a form similar to that favoured for highway bridges.

This guide offers an introduction to the design of steel and composite railway bridges, for readers who are generally familiar with the principles of limit state design. Some experience of bridge design, perhaps of the design of highway bridges, would be advantageous in understanding the detailed design procedures.

The guide begins with a review of the particular requirements for carrying railway traffic and interfacing with railway infrastructure. Since construction issues are particularly acute for replacement bridges, because of the need to minimise disruption to users of the railway, guidance is given on the constraints and options for construction.

So-called ‘standard’ bridge types were developed by British Rail from the 1950s onwards and although there is currently only one true standard bridge type in common use, features of many of the others are still appropriate. Section 4 presents a brief review of all the main forms of construction.

Whilst BS 5400 is currently the principal structural design standard, there are supplementary requirements that have been developed especially for railway bridges. A wide range of other standards that relate to track formation, clearances to railway traffic, etc. have been issued by Her Majesty’s Railway Inspectorate, Network Rail and London Underground. Section 5 catalogues these standards.

Design procedures are described in Section 6; there are three principal criteria for the design to meet – adequacy of fatigue life (railway bridges are subject to particularly onerous fatigue loading), adequacy of strength (i.e. at the Ultimate Limit State), and adequacy of the deformation response (which can affect the safety of rail traffic and the comfort of passengers). These procedures are described in Sections 7, 8 and 9.

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Success or failure of a design often depends on the quality of the detailing and durability often depends on the provision of features that allow proper inspection and maintenance. Section 10 provides a collection of 'good practice' details for the reader's guidance.

Finally, Appendix B presents a worked example of design calculations for a typical half through two-track railway bridge. The example illustrates many aspects of the design procedures described in the earlier Sections and should prove a valuable insight into the application of BS 5400 to the design of a steel railway bridge.

Most of the detail in the guide relates to (standard gauge) mainline railways and thus to the standard mainline loading adopted throughout Europe since 1973, following the demise of steam traction. The principles are, however, equally applicable to 'light rail' applications, although the different balance between static and live loading for such structures will affect design.

The guidance in this publication is intended to assist designers with the design of conventional rail bridges. Unusual forms of construction and temporary steel bridges are outside the scope of this publication

For a detailed commentary on BS 5400-3:2000, see the separate SCI publication *Commentary on BS 5400-3:2000, Code of practice for the design of steel bridges*[46]. References are made in the text to further general advice in 'Guidance Notes'. These are a series of notes, published by The Steel Construction Institute[47], that give concise advice on best practice in steel bridge design, from the members of the Steel Bridge Group, a technical group of experienced designers, fabricators and clients. References to specific Notes in that document are given in the form 'GN 1.02'. A full list of Guidance Notes is given in Appendix A.

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2 FUNCTIONAL REQUIREMENTS

2.1 Key functional requirements

There are two key functional requirements for a bridge carrying a railway:

i) Provision of satisfactory support to the railway traffic and infrastructure throughout the life of the bridge.

ii) Provision of adequate clearances between the structure and the traffic on and beneath the bridge.

The first requirement can be expressed in terms of requirements for:

- Strength and fatigue endurance
- Limiting the bridge deformation
- Robustness
- Durability

The second requirement is expressed in terms of various 'clearance gauges' that are defined by the railway and highway authorities.

To verify that the requirements continue to be met throughout the life of the structure (i.e. to ensure on-going serviceability), there is also a need to make provision for access to inspect and maintain the elements of the structure in a safe and convenient way.

To aid the appreciation of the functional requirements, Section 2.2 presents a simple overview of the nature of the railway infrastructure. The requirements for strength/fatigue endurance, deformation. robustness, durability and provision of adequate clearances are discussed in general terms in Sections 2.3 to 2.8.

2.2 The railway infrastructure

The railway infrastructure comprises the permanent way, which carries the railway traffic, access ways beside the track, and the associated plant, equipment and services that enable the railway to function.

2.2.1 Permanent way (track)

Permanent way, generally referred to as track, can be defined as comprising the rails, sleepers and ballast that provide support and guidance to rail traffic.

The great majority of the track on UK railways is of the conventional ballasted, cross-sleepered type. The rails are fixed to transverse sleepers (of either timber, steel or pre-stressed concrete), which are set in a bed of crushed stone ballast. New or replacement bridges are usually designed to accommodate this type of track, and the weight of the ballast usually adds substantially to the superimposed dead load. On many older steel bridges, the rails are fixed to longitudinal timbers, which are fixed directly to the bridge structure without any ballast. However, such construction details are now rarely used for new bridges on main line railways because of track maintenance problems, especially the run-on/run-off effects and the lack of flexibility in track location. See page 1 for further discussion on direct fastening.

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The standard gauge (i.e. the width between the inner faces or "running edges" of the rails) on the UK national railway network is nominally 1435 mm (4ft 8½ in). Since the railhead is typically 70 mm wide, the centrelines of the rails are 1505 mm apart, usually taken as 1.5 m when applying wheel loads in design calculations. Where there are two tracks, the minimum distance between the centrelines of the tracks is 3400 mm (see RT/CE/S/049[16], sheets A8.2 and A8.2). The space between tracks is commonly referred to as the 'six-foot'.

Rails

In the UK, until the 1950s, rails were mostly of the "bullhead" type, weighing typically 95 lb/yd. These are still widely found on London Underground, but are being replaced by the "flat-bottom" type. On the national network, bullhead rails have now been almost entirely superseded by flat bottom rails, weighing typically 113 lb/yd (56 kg/m). Heavier and deeper RT60 rails (also known as CEN60 or UIC60 rails), weighing 60 kg/m, have recently been introduced.

For new work, London Underground generally uses 113 lb/yd flat bottom rail running rails, plus third and fourth current rails of 150 lb/yd maximum.

Continuous welded rail

In the past, rails were laid with bolted fish-plated joints, typically at 18.3 m (60 ft) intervals. Whilst some jointed track remains, all main lines and most secondary routes are now laid with continuous welded rail (CWR), which has no fish-plated joints. The rails are pre-tensioned on installation so that they are stress free at a nominal temperature of 27°C. Any tendency for the track to buckle, owing to compressive stresses in the rails at higher temperatures, is resisted by the ballast and the weight of the track. Substantial "shoulders" of ballast are provided at the ends of the sleepers to ensure stability. Modern points and crossings also incorporate welded rails and are reinforced to accommodate the thermal stresses from the adjoining CWR plain line.

Where a length of CWR adjoins jointed track, special expansion joints (known as adjustment switches, expansion switches or "breather" switches) are provided. Similar joints are provided at the ends of longer bridges to accommodate thermal expansion/contraction of the bridge. On shorter bridges, no expansion joints are provided in the rails and the ballast accommodates any slight differential movement between track and bridge. The presence, or otherwise, of joints in the track will affect the way longitudinal traction and braking forces are transferred to the bridge structure (see Section 6.5.8).

On London Underground, the stress-free temperature for CWR is specified in Engineering Standard TE MTS 0314[29] as within the range 21°C to 27°C.

Sleepers and base-plates

Most sleepers are of pre-stressed concrete, which is preferred for heavily used high-speed routes, but some timber sleepers are in use, together with increasing numbers of steel sleepers. The rails are clipped directly onto sleepers, with a resilient elastomeric pad under the rail foot to prevent abrasion. On timber sleepers, the rail is clipped to a cast iron base-plate that is screwed or spiked to the sleeper. Standard concrete sleepers are deeper than timber or steel alternatives, but special shallow depth types are available for use on Network Rail bridges where construction depth is very limited. (These shallow depth concrete sleepers are not available for London Underground tracks.)

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Ballast

Ballast usually consists of hard angular pieces of crushed stone of about 50 – 65 mm size. It provides a resilient bed for the sleeper, distributes the load onto the bridge deck, and allows for drainage. It also provides a means of adjusting the track alignment and level. Normal practice is to provide at least 300 mm of ballast under the sleeper (230 mm for London Underground track), but it is sometimes necessary to provide less than this where construction depth* is limited. A minimum of 200 mm depth of ballast is necessary to prevent damage to the bridge waterproofing by track maintenance machines and to ensure the satisfactory distribution of wheel loads. (This minimum dimension is recommended in GC/RT5510, clause 6.3.6, regardless of sleeper type, although RT/CE/S/102[18] allows 150 mm below steel sleepers for ordinary track construction.) However, where construction depth is severely restricted, ballast depths of 150 mm (minimum) may be preferable to direct fastening, provided that the effect on load distribution is considered and appropriate precautions are taken to protect the waterproofing.

Where track is canted, the depth of ballast below the sleeper is usually measured under the low end of the sleeper but it may be acceptable in some circumstances to measure it under the low rail.

During normal track maintenance, the ballast is mechanically compacted under the sleeper. The compaction is not uniform under the length of the sleeper but is concentrated in the areas under the rails. The rules for distribution of load through the ballast[40] take this into account.

Track types, track depth

Normally the type of track to be provided, over a new or reconstructed bridge, will be specified by the owner of the infrastructure. Guidance on the types of track appropriate to different categories of lines on the national network is given in RT/CE/S/102[18].

For London Underground track types, refer to Engineering Standard E8001[25].

A summary of track types and their depths is given in Table 2.1.

Details of track components may be found in the *Track design handbook*[16].

* Construction depth is measured from tops of rails to bridge soffit, see Section 4.1.

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Table 2.1 *Types of track commonly found on the UK rail network*

Track type	Track depth*
Track for Network Rail	
CEN 60 FB rail on G44 concrete sleeper	377 mm
CEN 60 FB rail on type 600 steel sleeper	310 mm
BS113A FB rail on F27 or F40 concrete sleeper	368 mm
BS113A on EF29 shallow concrete sleeper	330 mm
BS113A on timber sleeper with baseplate	330 mm
BS113A on type 500 or 560D steel sleeper	281 mm
Track for London Underground	
BS113A rail on NTF 504 concrete sleeper	329 mm
Bull Head rail on timber sleeper	315 mm

* Track depth is measured from top of rail to underside of sleeper

Ballast retention

Bridges carrying ballasted track are constructed in the form of a trough with the sides raised sufficiently to contain the ballast, with some allowance for future track-raising during maintenance. Where tracks are canted, bridge structures are sometimes tilted transversely to avoid excessive ballast depths. However, this should be limited, as a tilted deck is likely to encourage migration of the ballast. A limit of 1 in 15 is usually observed for ordinary mainline traffic.

It should be noted that track maintenance methods tend to raise the track over time, since it is far easier to adjust the longitudinal profile by raising the dips than by lowering high spots. Furthermore, the tracks in many places were raised when CWR was introduced, to improve drainage, and wider ballast shoulders were provided. For these reasons many existing bridges have inadequate containment of the ballast (to meet requirements for new construction), particularly on the approaches at either end, where the original formation may be too narrow or the wing walls too low to support a ballast shoulder and cess walkway to current construction standards.

In designing new or replacement bridges, care should be taken to contain and support the ballast, particularly at the ends of the deck. Three-dimensional sketches or drawings will help in visualising the often-complex geometry and will also help in detailing the waterproofing, drainage, cess walkways, cable runs etc. in these areas.

Direct fastening

As discussed above, wherever possible new or replacement bridges are designed to accommodate conventional cross-sleepered, ballasted track. However, in order to achieve the minimum possible construction depth, or to minimise dead weight of a bridge, it may be necessary in extreme cases to omit the sleepers and ballast and to fasten the rails directly to the bridge deck. This is known as "direct fastening". Particular considerations are:

- It is difficult in practice to arrange and maintain a satisfactory transition from the directly fastened track to the ballasted track at the ends of the bridge. This is particularly true on high-speed routes, owing to the small permitted tolerances on track geometry, or where there is horizontal or vertical curvature.

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- Because track maintenance is highly mechanised, the introduction of short lengths of non-standard track, which require hand maintenance, increases costs.
- Successful installation and maintenance of a direct fastening system requires a high degree of care and precision that may not be achievable in the limited time and difficult conditions of a railway possession (for a discussion of possessions, see Section 3.2.2.
- For bridges with direct fastened track, consideration of noise levels may be required; special precautions can be taken if necessary. (See *Reducing noise emission from steel railway bridges*[44] and GN 1.06.
- The reduced dead load, compared to a ballasted bridge, may have an adverse effect on the dynamic response of the bridge, particularly for short spans, and a check should be made that the natural frequency is within the prescribed limits. Very light bridges can also be prone to hammering of the bearings under load, and are less resistant to accidental vehicle impact where they span over highways.

In general, a reduced ballast depth is often preferable to direct fastening from the point of view of track maintenance.

Recent strain gauge tests on rails in service indicate that local deck elements of direct fastening bridges need to be designed for much higher wheel loads than is the case for ballasted track, to take account of the effects of wheel flats (because there is no ballast to cushion such local effects).

Special baseplates are available that have been designed for use in direct fastening applications. These provide for limited lateral and vertical adjustment of the rail and also incorporate additional resilience to control bending stresses in the rail. Some types include noise-insulating material. Specialist advice should be sought from the railway authority for the design of transition arrangements at either end of the bridge.

The comments above are applicable to new or reconstructed bridges, particularly short span bridges, on the existing UK national rail network. Some of the disadvantages of direct fastening do not apply to completely new railways, or to rapid transit systems, and direct fastening is increasingly being used worldwide as new developments are made. Obviously, direct fastening is appropriate where slab track is in use instead of conventional ballasted track. (Slab track, consisting of a jointless reinforced concrete slab, to which the rails are directly fastened or are bedded into grooves, has been used in some countries but has not yet been used widely in the UK, except for tramways.)

2.2.2 Access along the railway

For the UK national rail network, document GC/RT5203[8] gives requirements for two types of access along the railway – 'cess walkways' for general access by permitted people and 'continuous positions of safety' immediately adjacent to the track, where people who have access to the line can seek refuge when a train comes. Minimum dimensions and clearances from the track are specified in the document. As a minimum, there should be a continuous position of safety on each side. A cess walkway (which is wider) can be used as a continuous position of safety, provided that the necessary clearances are observed; ideally such a walkway should be provided on both sides but it may be acceptable to have a cess walkway on one side and a continuous position of

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safety on the other. The railway authority will determine what is acceptable for any particular bridge.

2.2.3 Other infrastructure

Apart from the permanent way and access ways beside the track (see Section 2.7.1), the elements of infrastructure that may need to be supported include:

- Traction power systems
- Signalling cables and equipment
- Telecommunications cables and equipment
- Power cables
- Third party cables and pipes
- Mechanical and electrical equipment and plant.

In some cases, these elements may influence the structural form that is selected for a particular bridge.

Prior to detailed design work commencing, a survey needs to be undertaken by competent persons to identify the equipment present and develop proposals for its temporary and/or permanent relocation, if affected. These proposals may require additional possessions in advance of the main installation to minimise the work and the risk of unforeseen difficulties during the main installation possession.

Traction power systems

Traction power systems generally fall into two types:

- Overhead line electrification (OLE)
- Third and fourth rail systems, which are predominantly used in the south-eastern part of the UK and on London Underground.

Slewing or dismantling of existing OLE equipment should be avoided where possible and avoidance of such activities can have significant influence on the form of construction.

Steel bridges need to be electrically bonded to the traction current return rail or earth wire. Individual components may need to be electrically bonded together, if they are not adequately connected by bolting or welding. Longer bridges may carry masts supporting overhead electrification wires and the bridge will need to be designed to accommodate the forces imposed, including consideration of unbalanced forces if cables should break. Specialist advice should be sought as to how OLE supporting structures can either be relocated off the new structure or incorporated into the design.

The third and fourth rail systems have little effect on the design of the bridge, other than a slight increase in width in some cases, to provide clearance between electrical rails and structure.

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Cables

There is often significant cabling infrastructure running parallel to the tracks. The cables can be for signalling, telecommunications, power, third parties and other M&E functions. The cables are usually located in concrete toughing on UK main line railways and on brackets connected to parapet steelwork on London Underground. The cables can vary in age and condition; the latter will dictate how they are managed. It is quite common to be able to move the cables on to an adjacent temporary cable bridge, should there be adequate slack in the cables. After the bridgeworks, the cables are moved back, and tested before possession hand-back. Occasionally it is necessary to replace a section of cable in advance of the works, if its condition will not permit it to be moved or if there is insufficient slack for the cable to be relocated to its temporary and/or new alignment. Access to cables is controlled in a similar manner to that for the railway as a whole, and it is necessary to plan work well in advance, so that any permitted 'outages' for the cable services (i.e. when the service is interrupted) can align with the available enabling works possessions.

Other equipment

Other equipment such as signals and equipment cabinets are not usually located on or near structures. If they are, then specialist advice should be sought. In the case of a new bridge, if possible the site of the structure should be selected so as not to impact on these infrastructure elements, as these will increase the disruption to the railway by increasing the enabling works and requiring additional interfaces to be managed.

2.2.4 Services ducts

Provided that it does not obstruct the walkways, the usual precast concrete cable troughing can be continued over the bridge. Otherwise a special conduit may be incorporated in the structure, under or adjacent to the walkway, with removable access covers, and provision for drainage. Some thought should be given to the detailing at the ends of the bridge to ensure a suitable alignment and support to the cables (ensuring that any bends are large radius) and to provide a clear walking route without tripping hazards.

On London Underground, it is not usual for cables to be placed in concrete troughing. London Underground normally requires the provision of permanent cable brackets and supports on bridges.

2.3 Strength and fatigue endurance

Clearly, a bridge must have sufficient strength to carry the expected loading upon it. The loading depends on the type of traffic, speed of the trains and the number of tracks carried by the bridge. The loading and internal design forces for the structural members also depend on the geometry of the track (whether straight or on a curve) and of the bridge (particularly when the bridge is skew).

The loading on railway bridges is significantly affected by the dynamic effects of traffic actions, both due to the nature of the traffic and to the response of the structure. Particular aspects that give rise to dynamic effects are:

a) The rapid rate of loading, due to the speed of the traffic coming onto the structure, and the inertial response of the structure.

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b) The passage of successive axles or bogies, with approximately uniform spacing, which can excite the structure and, under certain circumstances, can create resonance.

c) Variations in wheel loads resulting from track or vehicle imperfections (including wheel irregularities).

d) Centrifugal effects on curved track.

e) Lateral wheel/rail contact forces.

f) Lurching of rail vehicles, which transfers some of the vertical load from one rail to the other.

Bridges must also be able to endure the repeated application of loading throughout the design life – they must be designed for fatigue loading. For railway bridges, fatigue loading is particularly onerous and may well govern many aspects of detail, the size of the principal members and even the form of construction. The fatigue loading depends, in addition to the above factors, on the annual tonnage of traffic on the line.

Requirements for strength and fatigue endurance are both ultimate limit state considerations. Guidance on structural design is given in Sections 6, 7 and 8. Guidance on loading, including the manner of allowing for dynamic effects in standard loading, is given in Section 6.3.4.

2.4 Bridge deformation

The following is a general discussion of functional requirements. Detailed limits on deformations are discussed in Section 9.

2.4.1 General

An allowance should be made for deformations and any pre-camber in the bridge deck when calculating clearances, headroom and construction depth.

Detailed requirements for deformation limits are given in GC/RC5510 and UIC Leaflet 776-3R[35]. The limits relate principally to vertical deflection under live load. There is an overall requirement that total deflection does not encroach on any clearances.

2.4.2 Dead load deflection

Consideration should be given to pre-cambering main girders to counteract dead load deflections (and to counter the optical illusion that a flat soffit appears to be sagging). However, on short spans this can cause the fabricator more work than is justified by the marginal aesthetic improvement. Recommendations are given in GC/RC5510.

Dead load deflection is an indicator of the natural frequency. Light, stiff bridges (high frequency) or heavy flexible spans (low frequency) can fall outside the limits of applicability of the design rules. Documents GC/RC5510 and UIC 776-3R give upper and lower limits on dead load deflection. It is a valuable simple check to evaluate dead load deflection at an early stage and thus ensure that the dynamic live load effects will be adequately covered by the dynamic factors included in the loading specification (see Section 6.3.4).

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2.4.3 Live load deflection

Limits on live load deflections are set in GC/RC5510 and UIC 776-3R[35] to ensure the safety of rail traffic and passenger comfort, to control rail forces and to ensure the track geometry remains within the specified limits. The limits imposed are onerous, particularly on high-speed lines and on multi-span non-continuous bridges , and should be checked at an early stage in the design.

For London Underground bridges, maximum train speeds are less than 100 km/hr and train weights are less; consequently, deformation limits are less critical.

Experience indicates that satisfying deformation criteria can dictate either structural form or the size of main structural members. Deformation limits that may have to be taken into account in the design relate to:

- Maximum vertical deflection in a span
- Twist (rate of change of cant along the track)
- Uplift at the end of a deck (kick-up)
- Change of angle at the end of a deck
- Horizontal deflection of a deck (change of angle at the end of the deck and lateral displacement in the span)
- Vertical deck acceleration (on high speed lines).

2.5 Robustness

Redundancy

Bridges should be designed such that they have sufficient robustness not to suffer damage due to accidental events or vandalism, to an extent that would be disproportionate to the severity of the cause.

Consideration should also be given to incorporating structural redundancy within the design, so that alternative load paths are available in the event of unforeseen failure of part of the structure. Generally, the structure should be designed so that the critical failure modes are those that give advance warning of failure (e.g. bending rather than shear).

General detailing

Railway bridges should be designed to withstand normal track maintenance, relaying and reballasting operations without damage. Waterproofing should be protected from ballast abrasion and damage by rail-mounted ballast tamping machines, tampers or hand tools by protective layers and adequate ballast depths. Girder flanges and any stiffeners adjacent to the track should be sufficiently robust to avoid damage during relaying operations or minor derailments. For this reason, and also to simplify waterproofing and inspection, web stiffeners of half through bridges are usually placed on the outside of the bridge.

Derailed trains

To comply with HMRI requirements[49], railway underline bridges should be provided with a “robust kerb” to contain derailed trains (see also GC/RC5510 Clause 7.3.3). For half through bridges, the main girders may be deemed to

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act as the robust kerb, provided that they are at least 300 mm above rail level, otherwise a separate upstand must be provided to fulfil this function. GC/RC5510 recommends that the kerb should preferably be set at least 1500 mm from the adjacent rail, although it is accepted that this is impractical in many cases, particularly in reconstruction of existing bridges. (GC/RC5510 also recommends that kerbs should be preferably 350 mm high, to allow for possible future track lifting.)

The most common type of derailment is where only a few wheelsets in a train are off the rails, each with one line of wheels between the rails and one running on the sleeper ends, and with the derailed vehicles upright and in line with the rest of the train. Such a partially derailed train can run for a considerable distance before it is brought to a stand and underline bridges should not introduce obstructions that would make the consequences of derailment worse. The area next to the track needs to be able to carry loads from derailed trains (see Section 6.3.5). Consequently, open type flooring is no longer acceptable for new or reconstructed mainline railway bridges.

For London Underground underbridges, reference to Engineering Standard E8001[25] and consultation with the Track Engineer is required to determine the provision of derailment containment.

Impact from traffic under the bridge

Bridges liable to impact from road vehicles (or impact from floating debris or vessels) should be designed to resist the appropriate loads (see Section 6.3.5). Elements liable to local damage in these circumstances, such as bottom flanges of girders, should be robustly proportioned and bearings may be required to provide resistance to uplift and lateral displacement.

2.6 Durability

Owing to the need for track possessions, road closures, or access through land belonging to third parties, it is usually difficult and expensive to gain access to railway bridges for the purpose of inspection or maintenance. Particularly for smaller bridges, the cost of access and inspection can be large in proportion to the value of the structure, or to the cost of any physical repairs. Minor defects can generate significant administrative costs in the form of reports, correspondence etc. especially where members of the public express concern, even though the defects may be of no structural significance.

From the maintenance engineer's point of view, the ideal steel bridge has a minimum number of parts to inspect, all of which are easily accessible for inspection, blast cleaning and repainting, without places where dirt and moisture can collect. From the assessment engineer's point of view, the ideal steel bridge will have all its main structural elements visible and measurable. Important components and connections that can only be inspected by removing the track and ballast should be avoided. Detailed bridge examinations take place on a different (and more frequent) cycle from that for track renewals and in practice it is difficult to co-ordinate the two. Track removal purely for the purpose of bridge examination is rarely justifiable, because of the disruption and costs incurred.

Protective coatings for steelwork need to be high quality and with a long life between maintenance, because of the difficulties and costs of access to the

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surfaces. Code of Practice RT/CE/C/002[11] gives recommendations for the selection and application of protective treatment and Specification RT/CE/S/039[15] gives the actual requirements for the treatments. Instead of surface coatings, in some circumstances (for example, deck type structural forms) the use of uncoated weathering steel may be appropriate and this can eliminate the need for any maintenance of the exposed surfaces during the life of the bridge (although periodic inspection will still be needed). General advice on the use of weathering steel is given in GN 1.07[47], in ECCS publication No. 81[38] and in the Corus publication *Weathering steel bridges*[39].

Steelwork should not be in direct contact with track ballast, because of the risk of damage and corrosion. Steel surfaces buried in ballast should be protected by waterproofing and protective layers or by concrete. Ingress of water behind protective concrete should be prevented by fully effective sealing. This is often achieved by the use of 'weather flats' (see detail in Section 10.2.5). For all main steelwork, other than fully encased beams, at least one face of each element should be accessible, so that the thickness can be checked with an ultrasonic gauge. For this and other reasons, half through bridges with a central plate girder located in the space between two tracks are undesirable, since the web of the centre girder may be inaccessible. If it is essential to provide a girder in this location, because of construction depth constraints, then it is better to provide two webs, with access between them, by using a box girder or a pair of Z-section plate girders. (Alternatively, provide ballast retention plates either side of the central girder, separated from the girder by an air gap sufficient to permit inspection and maintenance but detailed to prevent ingress of debris.)

It is considered good practice on smaller spans to use fabricated bearings as these can be more robust and require less maintenance than proprietary bearings; they are also generally more compact, which eases geometric constraints. Simple robust fabricated steel bearings can reasonably be expected to last for the design life of the superstructure. See Section 7.8 for guidance on design of bearings.

General advice on good detailing is given in the SCI publication *Design of steel bridges for durability*[45].

2.7 Clearances from the railway

2.7.1 General principles

Clearances between passing trains and the bridge structure, or any fixtures attached to it (e.g. cables supported off the parapet, handles on removable ballast plates, etc.), must be sufficient to ensure that:

- Trains and their occupants may pass safely (including passengers or crew who may be leaning out of windows).
- Staff may work on the lineside safely.
- Aerodynamic forces on the structure, trains and passengers are acceptably low.
- Where the railway is electrified, electrical arcing does not occur between the structure and the pantographs, collector shoes or other electrical equipment of passing trains.

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The boundary enclosing the necessary clearances (outside which all structures must be located) is known as the 'structure gauge'. The clearances required depend on the permitted speed of trains and track geometry. For new or reconstructed bridges, a simplified structure gauge is used, which is based on conservative assumptions for the sizes of trains and is defined by dimensions from the rails, rather than by clearances from trains. The requirements have evolved over the years to accommodate developments such as increased speeds, new rolling stock and larger freight vehicles.

The structure gauge and clearances are measured from the 'running edges' (or inner faces) and the tops of the rails. Where the track is canted, dimensions are measured parallel and normal to the plane of the top of the rails (see Section 2.7.4).

Minimum clearances are given in the HMRI *Railway safety principles and guidance*[49]. Part 2A, Chapter 6 gives the definitions and principles that are used to define a structure gauge. Further requirements and recommendations, based on the HMRI guidance, are given in the relevant Railway Group Standards[5,8,9,10]. For a full understanding, these documents should be consulted. The following Sections illustrate the application of current requirements to typical steel underline bridges.

The clearance requirements for London Underground tracks are given in Engineering Standard E8013[26]. London Underground operates two distinct sizes of rolling stock – 'Subsurface stock', which is very similar in section to national railways stock, and the smaller 'Tube stock'. Some open sections of Tube lines were originally constructed for subsurface stock; hence, they can still be used for new stock delivery, transfers between lines etc. Also, track maintenance contractors are increasingly using main-line rail-mounted machines on the Tube open sections where clearances permit. Consequently, clearances depend on the Line and the permitted rolling stock.

Outside the structure gauge, there needs to be space for access along the railway (see Section 2.2.2) and this requirement contributes to the total clearance necessary between track and structure.

2.7.2 Lateral clearances to parapets

For a simple composite deck type bridge, the lateral clearance between the track and the structure is as shown in Figure 2.1. The required clearance is shown as the sum of two components, one the clearance required to the near edge of the lineside access way and the other for the width of access way beside the track. The Figure shows the provision for a cess walkway, which would normally be provided on both sides of the line, but it may be acceptable to provide a cess walkway on one side and a (narrower) position of safety on the other. In this arrangement, the parapet upstand acts as a 'robust kerb' (see Section 2.5)

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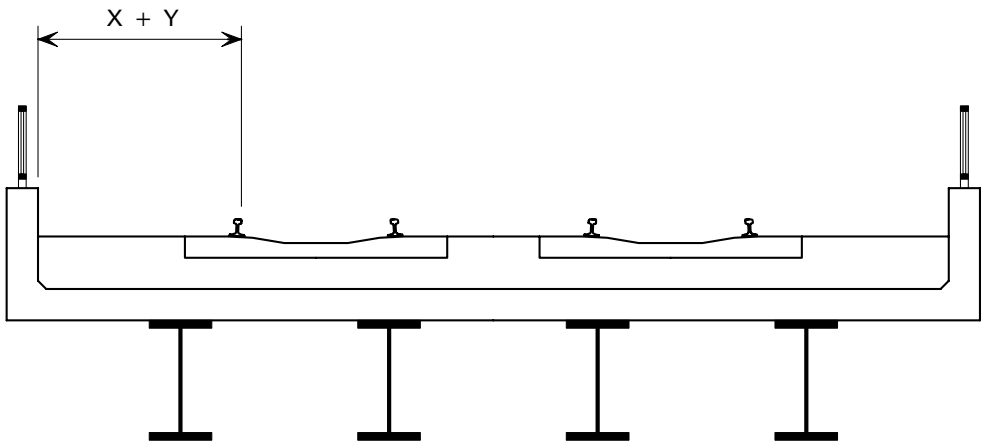

X *is the width of 'cess walkway' (700 mm minimum)*

Y *is the clearance from running edge of nearest rail to the walkway or place of safety (dimension depends on line speed)*
(These dimensions are defined in GC/RT5203)

Figure 2.1 *Lateral clearances to parapets*

For London Underground tracks, Engineering Standard E8013[26] gives the required clearances and defines the installation of 'Limited Clearance Warning Boards' where required clearances cannot be achieved

Walkway requirements for tracks are given in Engineering Standard E8052[27].

2.7.3 Clearance to girders of half through bridges

Half through construction is extensively used for steel railway underline bridges in order to minimise construction depth. The main girders are positioned either side of the track and the track is supported by a shallow floor spanning transversely between the main girders, near the level of their bottom flanges. (See descriptions of various forms of half through construction in Section 4.3.) To minimise the depth of the floor, the main girders are placed as close to the track as possible. In such cases, the positions of the girders, and the amount of clearance required between them and the nearest rails, depends on the height of the top of the girder above rail level. There are three basic cases, as described below:

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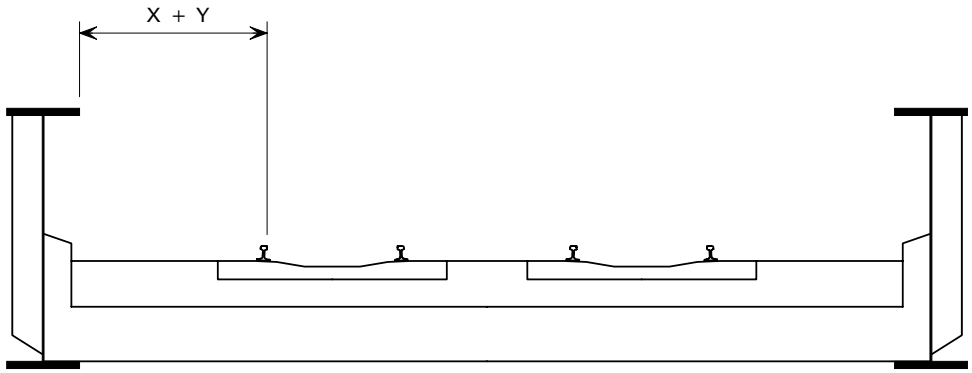
(i) Top of main girder more than 915 mm above rail level

Sufficient space must be provided for a walkway or place of safety between the track and the girder and this determines the spacing of the main girders. See Figure 2.2. The top flange of the girder acts as a 'robust kerb', as described in Section 2.5.

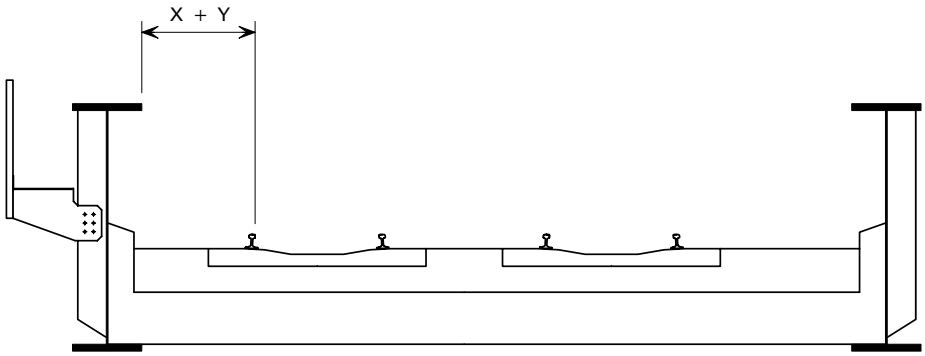


X *is the width of 'cess walkway' (700 mm minimum) or the width of a continuous position of safety (400 mm minimum)*

Y *is the clearance from running edge of nearest rail to the walkway or place of safety (dimension depends on line speed)*

Figure 2.2 *Half through bridge with top of main girder more than 915 mm above rail level and with an access way inside the line of the girders*

Where the width of a replacement bridge is constrained by the width of the existing abutments and there is insufficient width for cess walkways, separate walkways can be provided on the outer faces of the girders. There should still be positions of continuous safety alongside the track or a walkway outside the girders. See Figure 2.3



X *is the width of a continuous position of safety (400 mm minimum)*

Y *is the clearance from running edge of nearest rail to the walkway or place of safety (dimension depends on line speed)*

Figure 2.3 *Half through bridges with the top of the main girder more than 915 mm above rail level and with a walkway outside the line of the girders*

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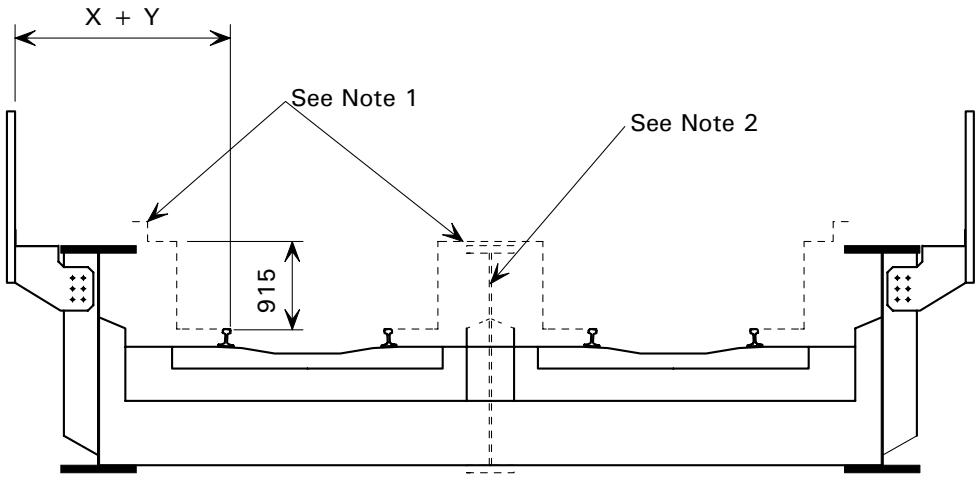
(ii) Top of main girder between 110 mm and 915 mm above rail level

When the top of the girder is between 110 mm and 915 mm above rail level, a walkway/place of safety is usually provided at the level of the top flange of the girder. The girder spacing is then governed by the lower sector structure gauge (commonly known as the platform gauge, since it is occupied by station platforms) given in GC/RT5212[9] Appendix 1. A typical arrangement is shown in Figure 2.4. In this Figure, a possible location for a third main girder is shown; it is located within the 'six foot', the space between tracks, and within the limits of the lower sector structure gauge. (See comments on durability aspects of the option in Section 2.6.)



X and Y are as defined in Figure 2.1

Note 1: The lower sector structure gauge (or platform gauge) as defined in GC/RT5212 Appendix I (note that the horizontal dimension depends on line speed and track curvature)

Note 2: The space available for a centre girder (not preferred)

Figure 2.4 *Half through bridge with top of main girder between 110 mm and 915 mm above rail level*

Where the top of the girder is more than 500 mm above the tops of the sleepers then access by steps or similar must be provided at intervals to give access from the track (See GC/RT5203[8]). Note that if the girder is less than 300 mm above rail level it cannot be considered to provide a "robust kerb" (see Section 2.5) to contain derailed trains. Consequently, the girder will need to be designed to carry vertical loads from derailed trains and a suitable parapet that is capable of acting as a robust kerb should be provided.

(iii) Top of main girder not more than 110 mm above rail level

When the top of the main girder is 110 mm or less above rail level, the girder can, subject to a specific derogation from Network Rail, be located within the outer part of the "area for items intended to come in close proximity to trains" given in the GC/RT5512 diagram for the lower sector gauge. This is shown in Figure 2.5. (Location of fixed infrastructure in this area was previously allowed by British Rail; Network Rail are currently considering altering the diagram.) In practice, the actual clearance from the running edge of the rail to the girder is limited by the length of the sleeper end and the need to leave space to pack ballast between the sleeper end and the structure (typically 120 mm minimum). Where there is third rail electrification, electrical clearance to the conductor rail may be the limiting factor (see the *Track design handbook*[16], Sheet A.8.7). Note that since, the girder is less than 300 mm above rail level,

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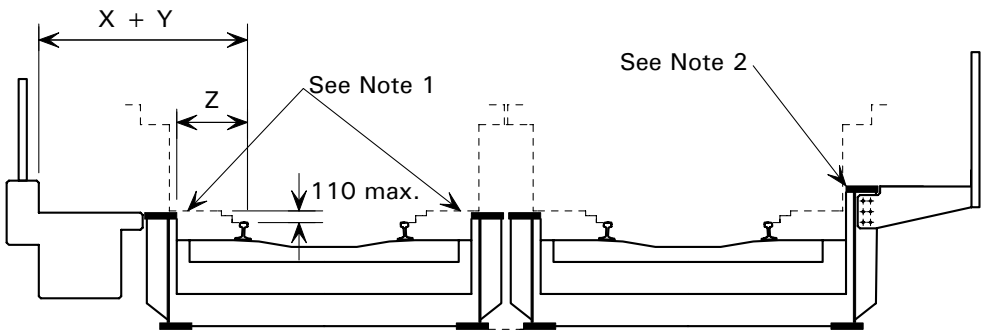
it cannot be considered to provide a "robust kerb"; the walkway must therefore be designed to support vertical derailment loads and be provided with a separate robust kerb, as shown on the left hand side of Figure 2.5. An alternative arrangement is illustrated on the right hand side of the Figure, where the outer girder has been raised to provide a robust kerb, allowing a cantilevered walkway to be used.



X and Y are as defined in Figure 2.1

Z = (Sleeper length – gauge / 2, plus sufficient gap to enable ballast to be placed at the sleeper end, plus an allowance for track maintenance and construction tolerances. For third and fourth rail electrification, additional clearance may be needed to the conductor rails.

Note 1: The lower sector structure gauge for areas close to the plane of the rails, as defined in GC/RT5212 Appendix 1. See further comment in text

Note 2: An alternative arrangement, with the outer girder raised to provide a robust kerb, permitting the use of a lightweight walkway

Figure 2.5 *Half through bridge with top of main girder not more than 110 mm above rail level*

On London Underground tracks, the relevance of the height of the top of girder above rail level depends upon the permitted rolling stock for a particular line, therefore reference must always be made to Engineering Standard E8013[26] and to its structure gauge and structure profile diagrams.

2.7.4 Effects of curvature and cant

End and centre throw

When a rail vehicle is on curved track, its corners overhang the rail to the outside of the curve by an increased amount. This increase is known as the "end throw". To the inside of the curve, the middle of the vehicle has an increased throw, and this is known as the "centre throw". Structure gauges are normally specified for straight track and the lateral clearances need to be increased on curves to allow for throw. Guidance on the calculation of throw may be found in the *Track design handbook*[16] and in document GM/RT2149[10]. The amount of additional width required to accommodate throw is small on normal radius curves and it is usual to provide a generous nominal amount. In the case of the structure gauge for the area up to 1100 mm above the plane of the rails, defined in GC/RT5212[9], Appendix 1, specific requirements are given for increasing the horizontal clearance to allow for throw.

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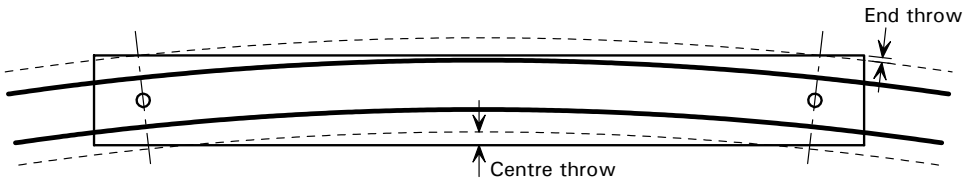
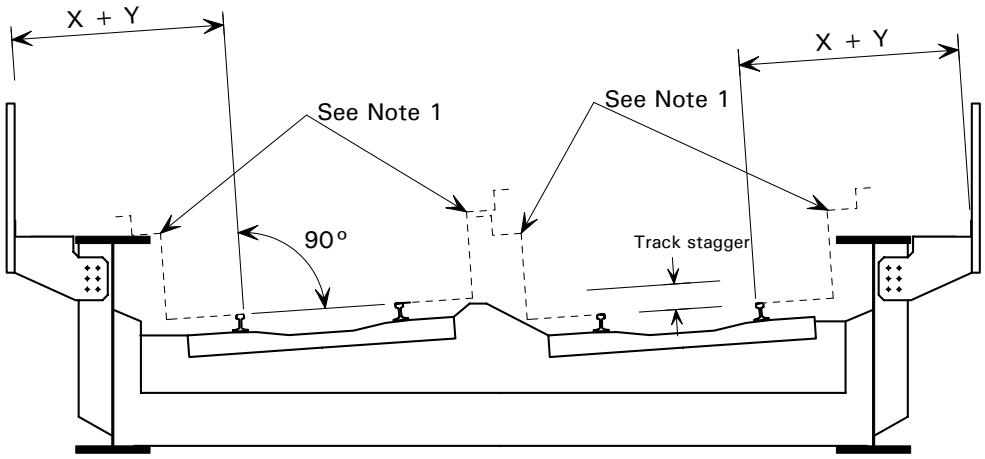


Figure 2.6 *End and centre throws for a vehicle on curved track*

Cant

Railway track on curves is usually superelevated to compensate for the effects of centrifugal forces. The amount by which the outer rail is raised above the inner rail is known as the "cant". The maximum amount of cant permitted is normally 150 mm, although in certain circumstances this can be exceeded. The effects of cant on clearances are usually dealt with by tilting the structure gauge, i.e. all clearances are measured relative to the plane of the canted track, rather than horizontally and vertically. This is illustrated, (applied to the half through arrangement illustrated in Figure 2.4) in Figure 2.7. Ideally, for track maintenance reasons, canted tracks are arranged coplanar but sometimes the track planes (top of rails) are staggered, as shown in Figure 2.7.



X is as defined in Figure 2.1

Y is as defined in Figure 2.1 plus an allowance for throw

Note 1: The lower sector structure gauge for area up to 1100 mm above the plane of the rails, defined in GC/RT5212 Appendix I (note that the horizontal dimension depends on line speed and track curvature)

Note 2: Any 'track stagger' will be limited by the railway authority, to ensure adequate ballast retention

Figure 2.7 *Half through bridge (tops of main girders between 110 mm and 915 mm above plane of rails) showing effects of cant on required clearances*

This approach is not entirely appropriate when considering lateral clearances to parapets based on considerations of staff safety, and the preferred method is illustrated in Figure 2.8.

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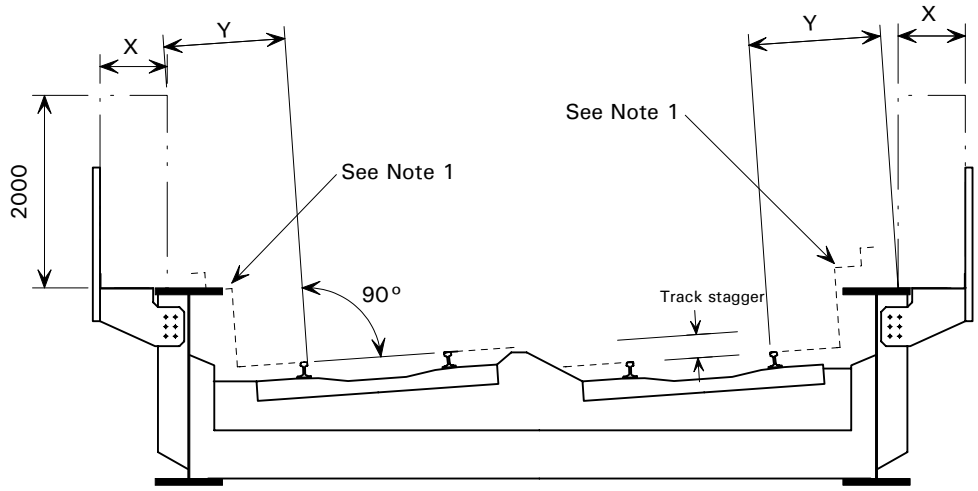

X is as defined in Figure 2.1

Y is as defined in Figure 2.1 plus an allowance for throw

Note 1: The lower sector structure gauge for area up to 1100 mm above the plane of the rails, defined in GC/RT5212 Appendix I (note that the horizontal dimension depends on line speed and track curvature)

Note 2: Any 'track stagger' will be limited by the railway authority, to ensure adequate ballast retention

Figure 2.8 *Half through bridge (main girder between 110 mm and 1100 mm above plane of rails) showing effects of cant (preferred method)*

On London Underground, the calculation of the effect of curvature (i.e. the end throw, centre throw and cant) on clearances is given in Engineering Standard E8013. This standard also covers the clearance approval procedure.

2.7.5 Track position on the bridge

The position of the track is determined by the railway authority. In determining the geometry of the bridge, the designer should take into account:

- The specified nominal position of the track
- The tolerance in track position permitted by the relevant railway standards (and whether such tolerance is already included in the clearance gauge)
- Any specified allowance for planned track realignment in the future
- Installation tolerance (typically ±25 mm on the plan position of the structure, see Section 3.7)
- If space permits, an allowance for other unforeseen adjustments (typically up to 100 mm)

Any proposals to change line or level of the track to suit the structure (even to meet current requirements for clearances, etc.) must be agreed with the railway authority.

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2.8 Underline clearances

2.8.1 Headroom

In the case of an existing underline bridge, the owner's legal liability for span and headroom is usually specified by the Act of Parliament under which that particular line was constructed. Before commencing design of a scheme for reconstruction, the legal status of the bridge should be established.

Many existing railway bridges over highways are of substandard headroom and are vulnerable to accidental impact from road vehicles. Where such a bridge is to be reconstructed consideration should be given to reducing the risk by increasing the headroom, by use of robust construction (see Section 2.5) and/or by providing collision protection beams or other measures, such as improved signing or traffic control. Detailed guidance is given in GC/RC5510[5]. Guidance on and requirements for the design of collision protection beams is given in BD 65/97[41] (part of the *Design Manual for Roads in Bridges*).

2.8.2 Construction depth

The 'construction depth' of a railway bridge is the vertical dimension between the tops of the rails and the bridge soffit.

Because the lowest acceptable soffit level is usually constrained by the existing roadway and its required headroom, and the level of the railway is very tightly constrained by track geometry, the available construction depth is a very important design parameter. It has a strong influence on the form of construction that can be adopted (see Section 4).

There are three components that add up to the construction depth of the bridge: the *track depth* (top of rail to underside of sleeper), the *ballast depth* beneath the sleepers and the *structure depth* (top of the deck or ballast tray to the underside of the structure and including the thickness of the waterproofing). Of these components, the track depth is fixed by the choice of track and the ballast depth is usually 300 mm (see discussion on page 1). The structure depth is therefore usually constrained by the difference between the available construction depth and the sum of track depth and ballast depth.

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3 CONSTRUCTION REQUIREMENTS

3.1 General

The design of railway bridges has always required the design engineer to give detailed consideration to the possible methods of construction that might be available at a particular site. This is considered a fundamental requirement in order to produce a design solution that can be translated into reality within the very short periods usually available for such activities. This is particularly so in the case of underline bridges because they are required to be capable of supporting the imposed railway loads by the time the structure is ready for reinstatement of the track.

With the introduction of the Construction, (Design and Management) Regulations in 1995, the need for the designer to consider carefully the effect of his proposals, from a safety point of view, was formalised. The designer is required to undertake design risk assessments and thereby develop a solution that minimises the health and safety risks to the public and the personnel who will build and maintain the bridge. For general guidance on the effect of CDM on steel bridge construction, see GN 9.01[47].

When the requirements of the CDM Regulations are considered together with the client's operational requirements, it becomes obvious that knowledge of methods of construction, and the suitability of each, play an important part in selecting the appropriate design solution for the particular site.

3.2 Minimising disruption to the operational railway

Minimising disruption to the railway is one of the key criteria to be understood by the designer in developing a successful solution. Due to the nature of the railway, access on or near the railway is restricted and generally personnel or operations that can lead to objects coming within a horizontal distance of 3 m of the nearest operational rail can only be undertaken during closures of the railway, called possessions. (The restriction applies also to plant that could potentially fall and land within this distance.) Even work outside the 3 m limit (such as preparatory works) may be subject to speed restrictions being placed on the line and this clearly is another disruption to railway operations.

Therefore, prior to commencing the design of a new or replacement railway structure, consideration must be given to the level of disruption that would be acceptable on the railway line(s) that would be affected by the works. For example, on a little used siding, a closure in the order of weeks may be acceptable but on a busy suburban or main line, a closure of only one or two days may be the maximum available for the proposed works.

Disruption can be considered in two main categories, 'Speed restrictions' and 'Possessions'.

3.2.1 Speed restrictions

Any works that are likely to affect the stability of the track and thereby increase the risk of a derailment occurring will require a Temporary Speed Restriction

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(TSR) to be imposed. Installation of the track and ballast on to a newly constructed bridge will require the imposition of a TSR. The severity and duration of the TSR will be governed by the amount of time available for the trackwork contractor to lay and bed down the track. Only a certain level of disruption is permitted on a particular stretch of railway line at a given time; this may dictate when the speed restriction can be available for particular bridgeworks.

3.2.2 Possessions

Most bridgeworks are undertaken during possessions. A possession can be defined as the closure of a section of the railway to normal rail traffic

The availability of possessions is classified as either 'Rules of Route', which are those available for the day to day maintenance of the railway or 'Outside Rules of the Route' (also known as 'abnormal'), which are special possessions usually of longer duration and booked for specific activities.

Rules of Route Possessions, available for regular maintenance of the railway, usually vary between 4 and 29 hours depending on the lines affected. These possessions are generally booked three months in advance. Sometimes it is possible to reduce this booking period by undertaking works on the back of a possession provided for other work, as long as it does not adversely affect the works the possession was originally booked for.

Outside Rules of the Route (abnormal) possessions are the ones most commonly used for bridgeworks because they generally offer a longer duration for construction activity. These possessions are usually booked up to two years in advance and are commonly up to two days duration or, in exceptional circumstances (such as over periods with public holidays), longer. Occasionally the abnormal possession duration for the main bridgeworks will have been agreed in advance of the design commencing, thus representing a constraint on the solution to be designed.

The possession duration includes for all activities, from taking the possession to handing it back. Depending on the complexity of the railway infrastructure, (such as the track layout, traction power and signalling at the particular location, most of which has to be removed and replaced to allow the bridgeworks to proceed), there is often little more than half the total length of the possession available for the bridgeworks. Therefore, this often leads to prefabrication being maximised and any connection details being robust and fully proven in advance of the installation possession.

Because of the importance of handing back the possession on time for train operations to recommence, the designer needs to produce a viable scheme that can be constructed within the available possession period, during any reasonably foreseeable inclement weather and with consideration given to the robustness of every detail. Specifying that all components are trial erected, and ensuring there are adequate tolerances in the design for fit up, will significantly help to achieve this.

It should also be remembered that any closure of a road below the railway will require the permission of the highway authority, although the requirements are not usually as onerous as those for railway possessions.

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3.2.3 London Underground

The situation is similar for London Underground, except that work can also be carried out during nightly engineering possessions. The period between traction current 'off' and 'on' is termed Engineering hours and it is often possible to carry out some structural work within this short period. Where the work would extend into normal traffic hours, a possession for the full duration of the works must be applied for.

3.3 General site constraints

For most structures, it is necessary to understand the impact of site constraints before developing the design solution, as these factors can dictate the method of erection and the form of the structure.

3.3.1 Site Access

This is the single most important consideration because access by road to the bridge site is not always available, particularly if the bridge spans over an obstruction other than a road (e.g. a canal, river, or flood plain). In such cases careful thought needs to be given to researching the types and quality of access that might be arranged to enable the particular design solution being considered. Examples of types of access often considered are:

- The construction of temporary road across an adjacent landowner's field or flood plain after appropriate negotiations have been conducted. Access across a flood plain is risky for obvious reasons and is subject to consent from the Environment Agency.
- Accessing the bridge site along the railway from one end using road plant if convenient access to the railway is available nearby.
- Accessing the bridge site along the railway from one or both ends using rail mounted plant or road-rail plant.

The quality of site access will determine the type and size of bridge elements (and the type and size of plant) that can be brought to site. Particular care needs to be taken to make sure that the load carrying capacity of any bridges supporting the access road are adequate for the weight of plant and bridge elements being considered.

3.3.2 Available working space

Many rail sites are in heavily built up areas and often buildings have been erected adjacent to the railway after the line was constructed. These may preclude various methods of erection and significantly constrain others. Apart from the obvious physical obstructions, there are the issues of complex logistical planning necessary to be able to work within a very small site that may consist of a single street. This may affect the required possession duration, with each bridge element needing to be delivered to the crane in a specific order in accordance with the programme.

Other issues include those of air rights when either lifting elements or working over others' property. Consideration should be given to these by the designer to try to minimise and eliminate these requirements where possible.

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3.3.3 Services

Services include all statutory undertakers plant, from sewers and fibre optic telecommunications cables buried in the road through to overhead power lines. As part of the initial option development, the location of utilities services need to be confirmed and their impact on the proposed scheme identified. This will need to cover the viability of any diversionary works together with an order of costs. These services can have a variety of impacts for example:

- Sewers and other older gas and water services with shallow cover may preclude access or use of heavy plant such as transporters or large cranes, unless additional protection and or strengthening measures can be implemented.
- Fibre optic cables may require diversion for strengthening of existing substructures, which may cost more to divert than the rest of the scheme.
- Overhead power lines may require diversion or isolation during the installation possession if a cranage scheme is being proposed.

3.4 Third party issues

Planning approval for the reconstruction of existing railway bridges is usually sought by the owner under a permitted development order. Usually the designer will be asked to prepare a drawing showing comparative elevations of the bridge before and after reconstruction, including details of the colour of the steelwork.

It is usually necessary to consider the effect of the works on statutory service authorities. Many railway bridges carry telecommunications cables owned by non-railway authorities, in addition to the railway-owned signalling, electrical supply and telecommunication cables.

Liaison with the local authority regarding the effects of the works on street lighting and road signs may be required. There may be constraints on the availability of lane/road closures owing to the lack of suitable diversionary routes, or the needs of emergency services or bus operators.

For waterway crossings, the appropriate river/canal authority must be consulted.

Local or residential noise and vibration restrictions may affect the design of the bridge or track form required, particularly on London Underground lines. There are various types of noise- and vibration-reducing track forms available, with direct and indirect fixings. The bridge construction may have to be designed to accommodate a particular specialised track form.

3.5 Methods of bridge erection

The erection of a new steel railway bridge will involve different activities depending on whether it is a completely new structure or the reconstruction of an existing superstructure.

The main methods of erection are as follows:

- Lifting (by crane)
- Rolling

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- Sliding
- Transporting.

The most commonly used method of erecting a steel railway bridge is that of lifting by crane, either piecemeal or in a single lift. However, in some situations bridge elements cannot be lifted into their final position because there is insufficient possession time available (for erection of pieces or for slewing of an OLE system) or because of limitations on crane reach. Under such circumstances, consideration needs to be given to designing a bridge that can be erected off-line and then, during a possession, be moved to its final position. In these situations, the other methods of erection such as rolling, sliding and transporting need to be evaluated.

Diversion of the railway to facilitate construction of a replacement bridge is unlikely to be feasible except in extremely rare cases where it is part of a major scheme and, even then, it would have to demonstrate considerable advantages over other methods. The disruption and costs of diverting a railway line, especially the specialist signalling, electrification and permanent way elements, are prohibitive and would probably outweigh the cost of rest of the scheme.

3.5.1 Lifting

Traditionally, the most commonly used method to erect a bridge is piecemeal by use of one or more cranes. An all-steel bridge fully fabricated in a workshop and trial erected to prove acceptable fit-up of the components can be speedily erected on prepared substructure. The actual amount of time for the erection depends on the overall size and type of the structure, particularly on the type of connections to be made, and on the type of crane to be used.

The cranes used are usually road-mobile but can also be either rail-mounted or on a floating vessel. Developments in both road-mobile and rail-mounted cranes have significantly increased the size of elements that can be installed. Some examples of typical scenarios are as follows:

- Short span bridges (up to about 17 metre span) can generally be designed, fabricated, and transported to site in such a way that complete bridge decks, each capable of supporting one track, can be lifted into position in a single operation. Two independent half through decks, such as the ‘Z’ or ‘U’ Types (see Section 4), can carry a double track railway. For a 10 m span, a single-track all-steel or steel/concrete composite ‘Z’ or ‘U’ type unit would weigh between 35 and 70 tonnes.
- If site constraints do not allow lifts of such magnitude, such as where OLE is present and there is not enough time to allow it to be slewed, then a short span bridge may have to be delivered to site as girders and short lengths of transversely stiffened deck. Having previously trial erected the deck complete with sill units at the fabricator’s works, the process of bolting together the components on site can be rapid.
- Medium span bridges, from say 15 m to 40 m, and continuous multi-span bridges of a similar configuration are usually too heavy or bulky to be lifted in a single lift and are therefore most easily dealt with piecemeal, as long as the construction is fully prefabricated, either wholly in steel or steelwork with precast concrete units.

In assessing the viability of cranage proposals, advice should be sought from specialist cranage contractors, especially for larger and more complex lifts using

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more than one crane or using vessels. This advice should also be sought to ensure a workable cranage scheme is developed that takes account of the physical dimensions of the proposed crane, access for the crane (including for rigging), the suitability of the ground to support the crane out rigger loads and the presence of services.

Before the final selection of cranage as the erection method, consideration should also be given to the following:

- Access to site for crane and bridge elements
- Overhead power lines
- Overhead electrification equipment
- Underground services
- Exposure to wind or flooding
- Available possession time
- Availability and locality of back up plant, spares and fitters.

Further general guidance is available from documents such as the Network Rail Model clauses that cover such works.

3.5.2 Sliding/rolling

As with the other techniques, there is little technical literature on this subject therefore detailed information on systems can only generally be obtained from specialist contractors and manufacturers of such systems.

The bridge designer does not need to detail the system (although, to comply with CDM at least one specific system should be shown to be viable) but information on the techniques is given here to aid an appreciation of the benefits and restrictions of such systems.

Although the principle behind the two methods is the same, sliding and rolling are different techniques.

Sliding consists of sliding the structure (usually heavier structures) on low friction surfaces. A large number of systems are available using different combinations of materials at the sliding interfaces such as phosphor/bronze or a PTFE sledge on stainless steel. The actual coefficients of friction will depend on the characteristics of the particular materials used. Typically, the coefficient of friction at breakout is likely to vary between a minimum of 5% and a maximum of 12%. During sliding, the coefficient generally reduces to between 2% and 8%.

Rolling consists of either rolling the structure (usually lighter structures) on ball bearings constrained in a channel or on proprietary rollers supported on rails or on proprietary skates. The comparable values for friction for rolling on 75 mm diameter steel balls is about 10% at breakout and 2.5% once rolling. For proprietary roller units these values can be as low as 2.5% for both breakout and rolling resistance.

Both techniques potentially carry higher risks and may be more expensive than erection by crane. The risks are generally associated with the installation of the temporary works to support the slide paths or methods of controlling the

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movement so that the structure does not crab and/or seize during the possession. However, these methods offer speed advantages and in certain circumstances offer the only viable method of erection. The principle of building the bridge in one place and moving it to its final position is particularly suitable for the erection of medium to long span and multi-span steel, steel/concrete composite bridges. Such bridges would take too long to erect in situ, unless a special long possession can be obtained.

Both methods require the construction of temporary works adjacent to and beneath the track to support the slide path on which the new bridge will be erected. The rails, rolling paths or sliding paths are usually built into the temporary works and generally run parallel to the lines of the substructure elements. They can in some cases form part of the permanent works. The bridge is then constructed parallel to its final position and slid /rolled into position, usually at a higher level and then jacked down onto the permanent bearings.

The location of the roll/slide path is usually inboard from the bearing positions and clear from the face of the abutment, to avoid congestion in the bearing shelf area. Consideration also needs to be given to how the structure is supported during the transferring operations, as some structures are more tolerant of differential settlement of supporting temporary works and varying support reactions than others. It may be necessary to interlink the hydraulic circuits to supporting jacks to ensure that the supporting reactions are evenly distributed and to avoid over loading individual sledges, rollers and jacks.

Depending on the weight of the bridge and friction resisting movement, the plant used for moving the bridge usually consists of a system of winches for the lighter structures or hydraulic jacks for the heavier ones. The system will also need to be designed to withstand a force normal to the main pulling force to ensure that it is suitably robust; this value is normally taken as 30% of the maximum pulling force. Provision should be made for reversing the direction of sliding or rolling, should there be an overshoot or if crabbing occurs.

Closed circuit television cameras are often used to provide information on progress so that action can be taken if required. This is particularly relevant if the chosen method is susceptible to seizing if one end of the structure moves differentially in relation to the other.

Due to the importance of completing the slide/roll within the specified possession, a trial slide/roll, typically of 2 m, is usually undertaken in advance of the possession to prove the system. Wherever possible, the complete slide track or roller path should be installed in advance. In exceptional circumstances, where this is not possible, it is essential that the construction of the slide track or roller path be simulated as closely as possible during the trial slide/roll, to validate this part of the possession programme.

From experience, a key consideration in determining the feasibility of a sliding or rolling method of installation is the positional tolerances that need to be met and whether they can be attained within the limitations of time and space during a possession.

It is possible to use sliding/rolling techniques also to remove the existing bridge so that it can be safely demolished without affecting the critical items of work.

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In this case, there must be enough space to accommodate a similar slide track or roller path on the opposite side of the railway.

3.5.3 Transporting

This is a relatively recent technique for moving railway bridges, first used to erect a new bridge in the early 1990s. The technique consists of using multi-axle highly manoeuvrable vehicles sometimes described as 'self propelled lifting vehicles' (SPLVs), to lift the ready assembled bridge from temporary works at a nearby site and transport it to its final position. This method of installation is particularly suited to bridges over highways or where the presence of overhead electrification or very restricted possession duration precludes crane erection. The method is generally more expensive than erection by crane and is comparable with sliding and or rolling.

SPLV units are typically 2.4 m wide and 8.4 m long and can lift 96 tonnes. The units can be interconnected to increase the lifting capacity. The SPLVs can move in any direction in the horizontal plane, and rotate 360 degrees about a vertical axis; they also have a limited capability to lift and lower the structure. The vertical movement range is generally limited by the stroke of the jacks on each wheel (typically 600 mm) and some of the available range will be utilised in compensating for unevenness of the terrain over which the trailers have to pass. On flat level ground most of the range may be available but on rough or uneven ground little may be available for raising/lowering the structure. Additional vertical movement can be provided using timber block towers on top of the vehicles, but removing the timbers is a time consuming operation and therefore this should be kept to the minimum practical. In detailing the structure, consideration should be given to the accuracy to which the structure can be positioned: a tolerance of ± 25 mm should normally be allowed for plan position of components.

The main considerations, when this type of method is being adopted, are:

- The surface on which the vehicles are to traverse. An existing highway is generally considered to be suitable, although the effect of the trailers on particularly weak or vulnerable surfaces, services and basements etc. should be checked. Unmade ground, especially newly exposed embankments when installing a new structure where no previous structure existed, or winter conditions, are less suitable for SPLVs because of the additional risks.
- The effects on the bridge while it is being transported need to be clearly understood and designed for. This is not only a capacity issue but also one of deformation (including twist), as the vertical reactions from the temporary support positions can cause horizontal displacements that cause difficulties with positioning the structure, particularly for high skew structures.
- Access is required to a suitable site where the bridge can be temporarily erected. This site needs to be near the bridge site, usually not more than 500 to 700 m away, to keep traffic disruption and any relocation of roadside furniture to a minimum.
- Specialist advice should be sought from the contractor offering the SPLV system to make sure the vehicles can traverse the route between the remote site and the bridge site.

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- The bridge should ideally be fully waterproofed with track and ballast already laid before transporting it in, order to realise the maximum benefit from the system.
- Consideration should be given to using the same system for removal of the existing bridge.
- Urban areas generally are less suitable for this kind of treatment because of the lack of suitable vacant site within easy reach of the bridge site and the presence of significant roadside furniture that will require temporary relocation.

3.6 Replacement of the permanent way

The permanent way will require removal and reinstatement by a specialist contractor at the beginning and end of the installation possession. There are various methods and types of specialist plant available for this work, which will depend on the length of track to be removed, its condition and the site constraints. In some cases, the bridge may be erected complete with track, which must then be joined to the track either end of the bridge.

Where there is third and fourth rail power system, this is usually removed and reinstated with the permanent way. Where there is an OLE system, it is common practice either to design the structure so that it can be slid/transported into position (so that the OLE is unaffected) or to design the structure in discrete elements that can be installed in ways that require the OLE wires only to be slewed a small amount. Removal and reinstatement of OLE is very time consuming and should be avoided wherever possible.

Other railway infrastructure will also require reinstatement during the main bridge erection possession or in subsequent possessions. As a minimum, time will generally be required at the end of the main possession to restore and test signalling track circuits.

3.7 Buildability

When designing and detailing a steel railway structure, proper consideration needs to be given to the practicality of fitting the structure together, assuming that the work may be done in inclement weather and at night. This leads to the development of details that can accommodate realistic site erection tolerances.

To ensure fit of the structure in the limited possession time available, all components should be fully trial erected at the fabricators works. This is usually done before the application of protective treatment and waterproofing but after all welding is complete (assuming all site joints are bolted, see discussion below). The trial erection should include placement of any precast concrete elements such as sill units and ballast walls and ancillary items such as ballast plates, as any lack of fit with these items is far easier and more economical to address in the fabricator's workshop rather than on site during the installation possession. For general advice on trial erection, see GN 7.04[47].

Whichever method of erection is used, it is best to minimise the site activities as far as possible. Activities such as waterproofing should ideally be undertaken before delivery to site. This usually means that they are carried out under controlled conditions. Obviously, when steelwork connections still have to be

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made at site there will be areas of steel that will have been masked and therefore require to be coated or waterproofed on site. These will be relatively small areas and can therefore be accomplished quite quickly. Pre-waterproofing the deck also removes a major weather-dependant activity from the critical path and this in itself is a major benefit.

The design should generally use shop welded and site bolted connections for steelwork, although there is a place for site welding in some circumstances (for example where pre-assembly is possible outside of possessions). Structural connections that require site welding during a possession should not be specified, as it is not easy to carry out welded connections to the specified quality in the limited periods of time available during a possession. However, bearing location plates are a particular exception, where the welded detail is not subject to fatigue loading and welding is specified to improve site tolerance.

Site bolting generally consists of using field bolts at key locations in the entire structure in order to pull the components fully together at each joint location, before replacing them progressively with the specified permanent fasteners. This ensures that the permanent fasteners will function satisfactorily as designed in the completed structure.

In situ site splicing of the main girders can be time consuming because temporary works in the form of trestles with working space for personnel at just the correct level are required near the splice positions. Such splices in box girders can be particularly difficult and, being on the critical path, should be considered carefully if possession time is limited. Where possible, splices should be made in advance of the possession, preferably at ground level.

Transportation of the fabricated elements to site also needs to be considered by the designer. For transport by road, structural components should where possible be detailed so that the loads comply with 'ordinary' size and weight limits. This conflicts, to some extent, with the desire, for reasons of durability and maintenance, to minimise the number of site connections and thus the number of pieces that are transported. Also, the limited times available for erection often dictate the use of the largest possible component sizes and thus the transport of 'larger loads. Larger loads are termed 'abnormal indivisible loads' and movement requires notification to the police. Very large loads require special 'Movement Orders'. See GN 7.06 for more detailed advice.

Transport by rail should generally be avoided, if at all possible, because of the risk it imports into the construction activities. If rail transport is considered, specialist advice should be sought to ensure that members fit within the relevant loading gauge, which will be dependent on, among other things, the selected transporting vehicle type.

Generally, the following checklist is suggested:

- Check requirements for transportation of large and heavy loads
- Ensure that the size and weights of prefabricated units are appropriate to the capacity, reach and jib height of readily available cranes that will suit the site.
- Make site setting out simple and provide generous tolerances for location of the structure. (Typically, when erecting piecemeal by crane all bearing fixing details should accommodate tolerances of at least ± 25 mm in plan

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position, ±10 mm in vertical level and be able to accommodate lack of fit in bearing surfaces.)

- Provide prefabricated components with purpose-made lifting points symmetrically disposed about the centre of gravity and at same level, allowing units to be lifted and balanced easily.
- Carry out a trial erection of the bridge components at the fabricator's works.
- Do not make assumptions about critical parts of existing structures that are to be reused (for example, the thickness of the existing bridge abutments should be checked by coring or trial pits). Investigate thoroughly.
- Consider carefully the method of installing the bearings. RT/CE/C/008[12] provides detailed guidance notes.
- As far as possible, avoid the use of methods and materials requiring precise quality control or which are susceptible to bad weather.
- Avoid minor operations on the critical path for erection.

Further guidance on buildability is given in CIRIA Report 155[53]. The model specification document RT/CE/C/008[11] also includes guidance on a variety of railway bridge installation methods.

For a bridge replacement, seizure of the bearings, as a result of corrosion etc. could result in the existing deck structure acting as a prop between abutments. Stability checks on the abutments should be considered for each phase of construction.

Where cranes are positioned on ground immediately behind existing abutments, to lift in the replacement bridge girders and deck, the stability of the abutment under the erection loading must be considered.

Dealing with services

Where an existing bridge is to be reconstructed, the method and sequence of dealing with the existing cables during the works should be considered at an early stage. The expense and possession time required to disconnect and reconnect cables can be considerable, particularly where fibre optic cables are present. Temporary slewing of the cables onto temporary supports is usually preferable to disconnection, but the practicality of this depends on the amount of slack available, the condition of the cables, and whether they are on the outside or the inside of a curve. The presence of a temporary cable bridge may obstruct crane working, and affect the size of crane required.

London Underground lineside services (high and low voltage cables, signal and communication cables, compressed air mains plus third party cables) are generally supported across bridges off the parapet steelwork. Diversion and reinstatement for a bridge replacement is such a significant cost of a project that alternatives to roll-in and cranage require consideration. The use of SPLVs (see Section 3.5.3) to transport a complete bridge deck under cable runs, supported by a temporary transfer bridge, has been successfully employed.

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4 FORMS OF CONSTRUCTION

4.1 Influences on form of construction

In addition to the influences on bridge design discussed in Sections 2 and 3, there are three basic sets of parameters that determine the form of construction of a railway bridge:

- Available construction depth
- Span and geometric configuration
- Limitations imposed by the substructure

Construction depth

For some new bridges on new railway alignments, construction depth is not particularly constrained; the track level and the road level beneath can be fixed at levels that suit the structure. In such cases, there are more options to choose the form of supporting girders and deck for maximum efficiency and economy, and for aesthetic considerations.

Replacement bridges are much more likely to be constrained to a shallow construction depth, because of the need to maintain a clearance below and to avoid lifting the track. For very short spans, deck-type structures can be entirely arranged within a shallow construction depth but in many cases the only way to support the track is to arrange a shallow deck spanning transversely to longitudinal main girders either side of the track. This form is known as 'half through' construction or, for top-braced deep trusses over longer spans, 'through construction'.

Span and geometric configuration

Clearly, the span has a direct influence on the depth of the main girders of a bridge and thus on whether the girders can be arranged within the available construction depth.

Single span bridges have historically been very common and even multiple span bridges have tended to be built as a series of individual spans. There are many good reasons for this but the discontinuity at intermediate supports has often led to corrosion and maintenance problems. Today continuous construction is preferred, where possible, but this then requires consideration of the interaction of bridge and track (in response to longitudinal loading and thermal effects) and the consequences on the reactions at supports; see discussion in Section 6.4.4.

Many bridges span skew to the abutments that support them. This can give rise to track twist problems and difficulties in detailing the end cross girders and in arranging bridge articulation. On new bridges, there may be scope for building abutments square to the span, even when the railway crosses the road or river at a skew, but replacement bridges usually have to be built to suit the existing arrangement of the substructures.

Limitations imposed by the substructure

In addition to the limitations on replacement of existing skew bridges, replacement on an existing substructure often constrains the width of the bridge. The strength and form of construction of the abutments and intermediate

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supports are likely to have a strong influence on the detailing of the bearings and sill beams.

4.2 Shallow deck-type bridges

There are three forms of shallow 'slab' or 'plank' construction (i.e. where the deck acts mainly as a beam spanning between abutments):

- Solid steel slabs.
- Orthotropic deck.
- Steel beam sections encased in concrete (filler beam construction).

4.2.1 Solid steel slabs

The simplest bridges comprise simply supported slabs spanning longitudinally between abutments. Solid steel slabs can be used to form such decks for railway bridges up to approximately 3 m. Their advantage is a very low structure depth, much less than alternatives such as prestressed concrete. The inefficient use of material is offset by the low cost per tonne, due to the minimal fabrication required.

The slabs simply sit on the abutments on elastomeric strip bearings. Transverse continuity is not normally provided. To provide parapet supports and robust kerbs, independent reinforced concrete units spanning between abutments are often used, as shown in Figure 4.1. These provide the robust kerb and should be designed to carry the loads from a derailed train.

Traditionally, quite wide slabs have been used, with typically two 1.7 m wide slabs per track (this width corresponds approximately to half the normal centre-to-centre spacing between tracks). The slabs are 200 to 250 mm thick and no fabrication is involved, other than cutting to size. Waterproofing is provided on the top surface of the slabs; at the joints the waterproofing usually cracks in service but some leakage is normally accepted.

Usually, serviceability limits on deflection and rotation (see Section 9) govern the design and steel stresses are relatively low. In the absence of welding, fatigue is not normally a problem.

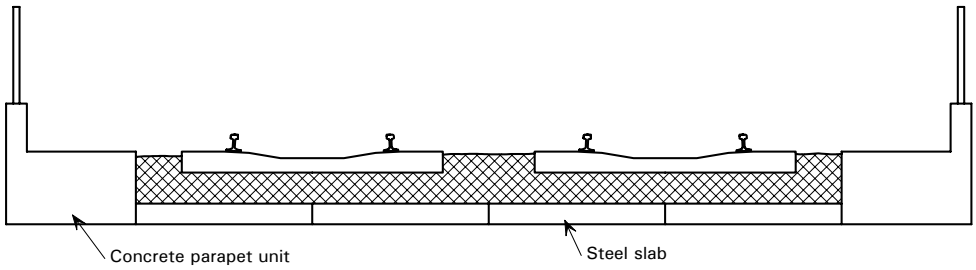

Figure 4.1 *Simplified part cross-section of a solid steel slab deck superstructure*

4.2.2 Orthotropic deck

For spans of up to 9 m, a very shallow structure depth of approximately 300 to 400 mm can be achieved using all-steel units spanning longitudinally between abutments. (For spans beyond the capability of solid slabs, orthotropic decks offer the shallowest structure depth of any form of deck type construction.)

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The deck units comprise a steel deck plate (typically 20 to 25 mm thick) with structural Tee sections welded to its lower face, usually at about 600 mm spacing. The Tees usually sit on elastomeric pad or strip bearings and end diaphragms are created by casting concrete around the Tees at the ends.

The deck units are relatively flexible transversely. Parapet/robust kerb containment can be achieved using independent parapet walkway units located clear of the tracks, in the same way as for solid steel slabs. See Figure 4.2

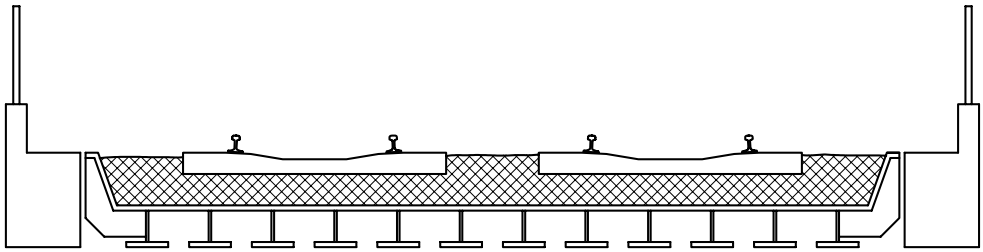

Figure 4.2 *Diagrammatic cross-section of an orthotropic deck*

Alternatively, parapets and robust kerbs can be provided by bolt-down steel units at either side of the deck. To resist the effects of the horizontal loads on robust kerbs, transverse bracing must then be provided between the Tee sections (see Figure 4.3).

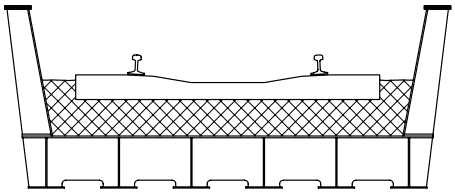

Figure 4.3 *Orthotropic deck with robust kerb*

Orthotropic deck construction is also used spanning transversely as the deck of some half through bridges (both with plate girders and with the BR standard box girders). See discussion about standard box girder bridges in Section 4.4.

4.2.3 Filler beam construction

Filler beam construction uses rolled steel sections embedded within and generally acting compositely with a reinforced concrete slab. See Figure 4.4. The concrete is cast to the level of the top of the bottom flanges and covers the tops of the sections.

To achieve transverse continuity, the bottom layer of reinforcement is threaded through holes drilled through the webs of the rolled sections and the top reinforcement is above the steel sections. Structure depths vary from a minimum of about 400 mm. In the UK, spans up to about to 13 m have been constructed and for 13 m span, UC sections can be used (the largest UC sections are about 400 mm deep). In Europe, spans up to about 30 m have been constructed; for these, European wide flange beams (HE and HD series, many of which are similar to UB sections) are used.

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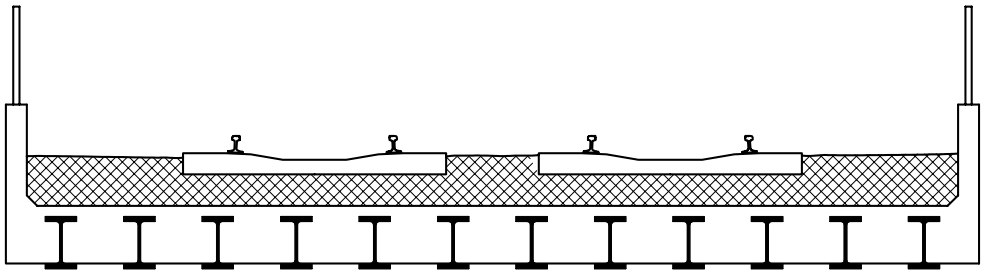

Figure 4.4 *Filler beam construction*

The deck can be poured in situ, if construction is outside a possession, but usually the deck is constructed in a number of units, which are precast and connected together using shear-key joints. Non-keyed joints may be acceptable (either leakage of water through the joints would be accepted or there would be a suitable sealing joint bridging adjacent units). Joints between units are normally arranged between tracks (i.e. the unit would be the width of track plus half the track spacing either side). Complete decks can be precast for installation as single units, where appropriate access and cranage are available.

Filler beam construction is also used transversely as the deck of some half through bridges. See further discussion below.

4.3 Half through plate girder bridges

Half through bridges are able to achieve shallow construction depths over relatively large spans and various forms have been widely used for railway underbridge construction. Essentially, such bridges comprise a pair of plate girders that span between abutments and a deck, spanning transversely between the plate girders. The deck may be of composite construction or an orthotropic deck. The lateral stability of the top flange (which is in compression) is achieved through U-frame action (see Section 7.1.3). The bridges are usually single spans, although continuous construction has been employed. To carry two tracks, a variation using three plate girders has been developed – see further discussion below.

British Rail developed a series of standard half through bridge types, some of them now outmoded but many of which have led to modern derivatives. This series of bridge types is described below, although not all details would now be acceptable for replacement bridges.

4.3.1 Z Type

The Z type evolved from the A type bridge, which was used between about 1950 and 1970. The A type comprises two I-section plate girders spanning longitudinally and simply supported on the abutments with a deck spanning transversely between the main girders just above bottom flange level. Cross girders are UC sections 152 mm or 203 mm deep at 600 mm spacing and with a concrete infill that is flush with the flanges; the ends of the cross girders are connected through a 4-bolt shear plate detail. Each deck supports one track; where there are two tracks, the arrangement is as shown in Figure 4.5. The span range for this type was 6 to 15 m.

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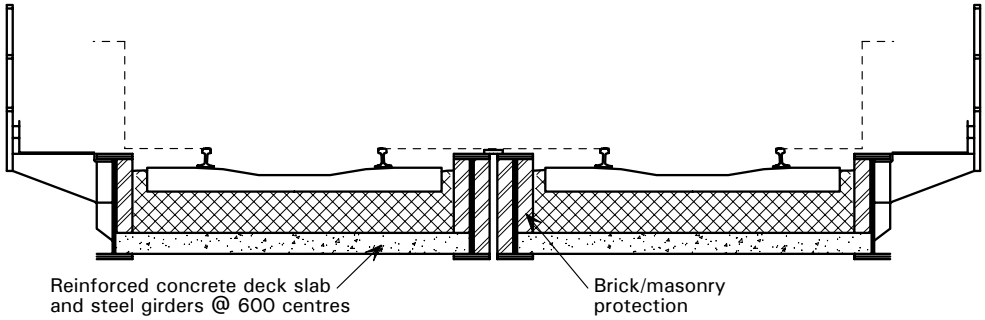

Figure 4.5 *Simplified cross-section of the former A Type decks*

The main problem with the A Type was that when two decks are provided to carry adjacent tracks, the gap between the flanges of the girders in the 'six foot' (the space between adjacent tracks) is too small to permit inspection and maintenance. For this reason, the webs were often protected by brickwork for their full height between the flanges of the main girders (although brickwork has not proved successful in preventing corrosion). Brickwork was also usually provided to the trackside faces of the webs to provide additional protection to the waterproofing and steelwork.

The Z Type deck which developed from the A Type solved the issue of inspection/maintenance access to the 'six foot' girders by offsetting their flanges (hence "Z") to create space between the bottom flanges and webs for access, but maintained a narrow gap between the top flanges. A filler is usually provided at the top flanges to close the gap.

The deck is a modified filler beam type construction (but see comment in Section 7.3 about the design of the deck slab) and the cross girders are bolted to the web through an end plate connection detail. The slab is reinforced transversely and longitudinal reinforcement is provided to control cracking. Structure depth is the depth of the slab (typically 325 mm in the standard details, which use HEM 160 sections), plus the thickness of the bottom flanges (which can add up to about 120 mm, including doubler plates).

The web stiffeners of the main girders are on the outside and connected to the top flange but not to the bottom flange. Walkways are usually provided as shown in the Figure, supported by cantilever brackets bolted to the main girders.

The standard design developed by British Rail and Railtrack for the Z Type is available as a set of standard drawings and a design manual[56]. The latest design was issued in 1996 (i.e. prior to the latest revision of BS 5400-3). The deck was designed fully but the drawings only provide details for the main girders: the actual girder sections have to be designed to suit the actual span geometrical constraints.

In the 1996 form of the Z type, the top flanges of both main girders project above ballast level, but do not exceed 110 mm above rail level, thus minimising the structure width (see Figure 4.6). However, current requirements for the provision of a robust kerb result in the outer girders being made deeper, to extend at least 300 mm above rail level. They are thus deeper than the inner girders and this in turn requires the deck to be widened on the outside.

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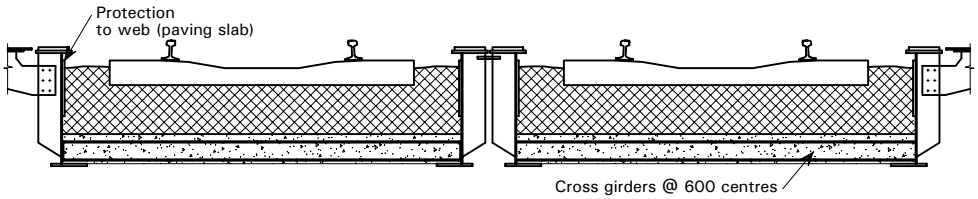
Every third cross girder is connected through a 6-bolt shear plate detail to form U-frames with the stiffened web; the other cross girders have a 4-bolt connection. The span range for the Z type is 6 to 17 m.



Note that the outer girders are now required to be deeper, extending to 300 mm above rail level

Figure 4.6 *Simplified cross-section of (1996) Z Type deck units*

Track with a curved horizontal alignment is usually canted. For low values of cant the variation in ballast depth is not a problem and Z Type decks are installed with the soffit horizontal. As the cant increases, the height of ballast to be retained may approach the top of the girder on the outside of the curve. To avoid this effect, the decks are often canted to match the inclination of the track.

The decks are normally simply supported on fabricated steel line pedestal bearings, as detailed on the standard drawings. If the decks are canted, the bearings are either increased in height under the outer girder, or located on bearing plinths with levels adjusted to suit the deck inclination and geometry. The standard details also include restraints designed to resist soffit collision loads from highway traffic where headroom is less than 5.7 m.

The deck are normally assembled and concreted prior to installation as single units. Smaller span decks have been completed at the fabricator's premises and delivered to site; others have been constructed in an adjacent site compound. Crane installation is the most common method, but transporters have been used on a number of projects, particularly where overhead electrification systems are present.

4.3.2 London Underground's 'modified A type'

On London Underground, the practice for half through bridges with minimum construction depth is to utilise a reinforced concrete deck the same depth as the cross girders, very much like the A type. However, intermediate web stiffeners are provided on the inner faces of the webs. At U-frame positions, the cross girders are bolted to the stiffener endplates as well as to the main girder webs. Haunches are provided against each web to contain track ballast and protect internal stiffeners; the haunches are cast integral with the deck.

4.3.3 U Type decks

The U type is a development by Cass Hayward and Partners of the Z Type. The concept replaces the main girders and cross girders with a single piece U-shaped fabrication, usually with a concrete slab cast on top of the steel floor to create a composite structure.

The steel floor plate of the composite U type is stiffened transversely with upstanding ribs and shear connectors are welded to the plate. The floor plate acts as the tension flange of the main girder. See Figure 4.7. Because there are few external stiffeners and the soffit is completely free of any stiffening and attachments, maintenance requirements are significantly reduced. The outer

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girder is deep enough to act as a robust kerb and, where there are two tracks, the inner girder is kept below the lower structure gauge.

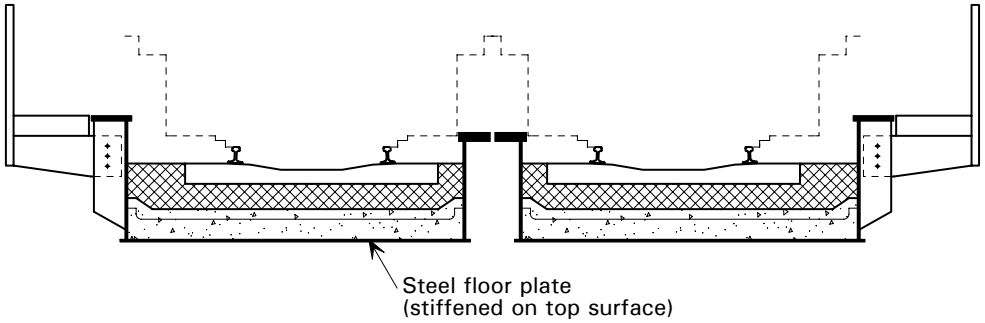

Figure 4.7 *Cross-section of a composite U Type deck*

The structure depth is about 220 - 230 mm and the span range is similar to that for Z types, i.e. 6 to 17 m.

4.3.4 B and C Type decks

For spans beyond the Z/U Type range, the depth of the main girders needs to be increased and this can be done by increasing the girder spacing, whilst keeping the structure outside the lower sector structure gauge (see Section 2.7.3 and Figure 2.4). The top flange level is usually more than 300 mm above rail level and can be considered to act as a robust kerb to restrain derailed vehicles.

This form of bridge is known as the B type (with two main girders, for single tracks) and the C type (with three-main girders carrying two tracks). The top flange width of the central girder of the C type is restricted by the need to maintain clearance to both tracks. Where U-frames are arranged under both tracks, the design of the cross girder connections at the centre girder needs to provide for significant end fixity and the main girder web and stiffeners need to cater for stress reversals induced by the U-frames as the tracks are alternately loaded. Alternatives to the double U-frames include an arrangement of 'L-frames' under each track (with no moment connections to the centre girder and that girder designed without intermediate restraints) and a combination of U-frames under one track and L-frames under the other.

Both types use a similar floor construction to the Z type (although with deeper cross girders for the wider span). The span range for these types is 12 to 23 m for the B type and 9 to 20 m for the C type. Complete bridges can be installed as single units by sliding or transporter methods.

The C Type has some disadvantages and two-girder forms are preferred where possible. However, for replacement bridges the constraints of, for example, avoiding major change to the loading on the abutments may necessitate use of the C type.

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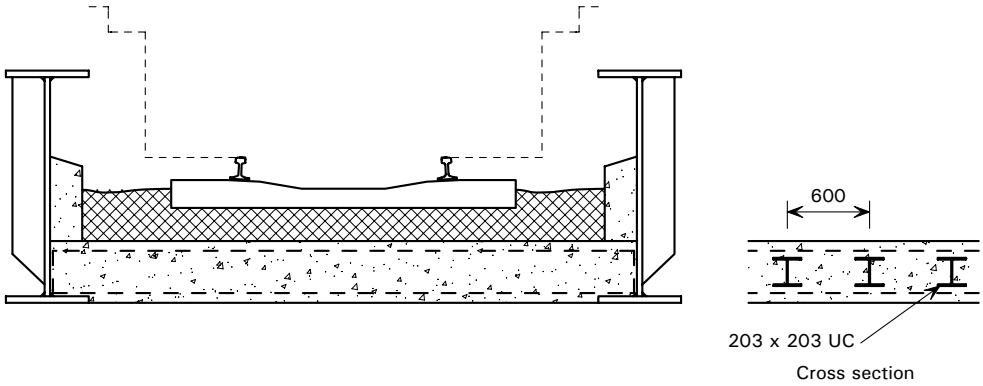

Figure 4.8 *Simplified cross section of a B Type deck*

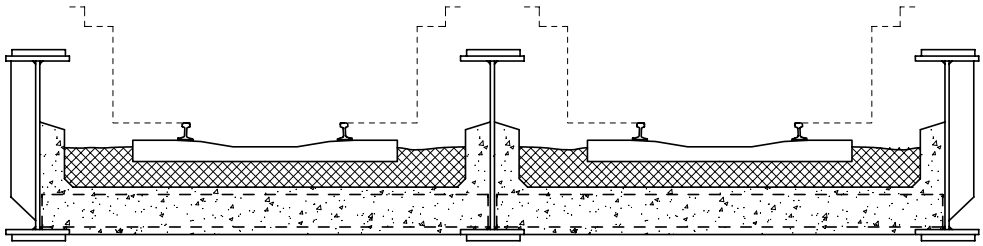

Figure 4.9 *Simplified cross section of a C Type deck*

4.3.5 D and E Type decks

The D and E type decks were developed by British Rail for larger spans. The height of the main girders exceeds the platform gauge and the width between is further increased to provide horizontal clearance to accommodate the tracks and a safe cess walkway at track level (see discussion in Section 2.7.3(i) and Figure 2.2). With these arrangements, spans up to 50 m (D type) or 45 m (E type) can be achieved.

The form of the floor depends on whether structure depth is critical. A shallower depth is obtained using cross girders at 1.5 m centres and with the top flanges encased within the slab; the slab is haunched down to bottom flange level. Haunching helps to reduce the deck self weight, but also pushes the bottom concrete soffit and longitudinal reinforcement higher, which limits the effects of global longitudinal bending on crack widths. Structure depth is about 500 mm (plus the bottom flange of the main girder) for the D type and about 600 mm (plus bottom flanges) for the E type.

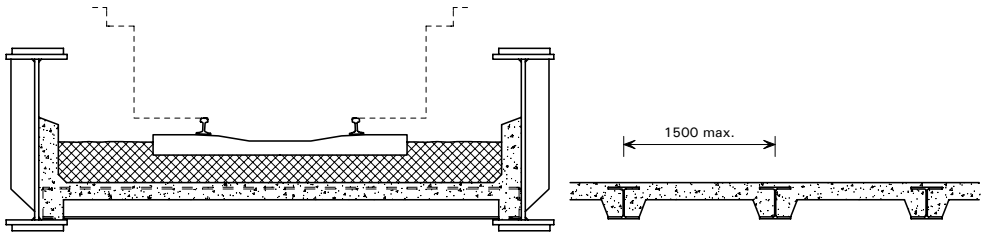

Figure 4.10 *Simplified cross section of a D Type Deck*

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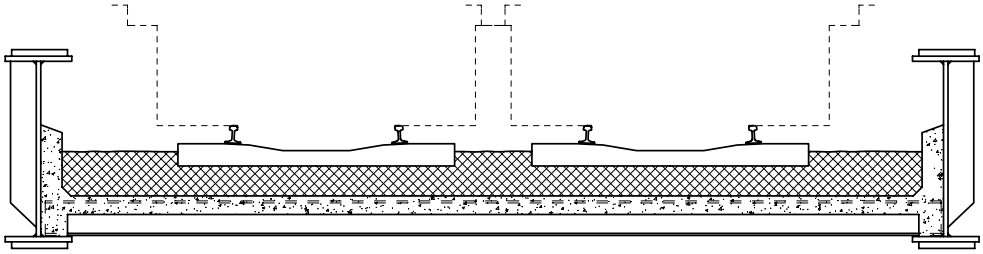

Figure 4.11 *Simplified cross section of an E Type Deck*

The bottom reinforcement in the slab is threaded through holes in the cross girder webs. See Figure 4.12.

When the D and E types were developed, the cross girders were connected with shear plates and a 4-bolt endplate, which minimises erection time during possessions but is comparatively inefficient for U-frame stiffness. A deeper connection with stiffened end plates provides a more rigid U-frame.

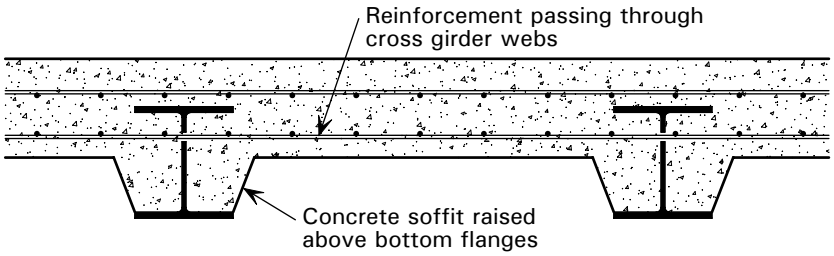

Figure 4.12 *Simplified part longitudinal section of D/E Type deck floor*

There have been numerous developments of the E type, using different deck slab constructions and also using an all-steel orthotropic deck (see Section 4.3.6).

One alternative that has been used where construction depth is not so critical uses UC section cross girders at up to 3 m spacing with a slab on top, acting compositely. This gives maximum economy for the slab construction, although the slab is quite heavily reinforced longitudinally, to span between the beams.

Two-track half through bridges with the main girders reduced in height to fit below the platform gauge limits and external walkways similar to the B Type single-track decks at top flange level are often also referred to as 'narrow' E Type decks. Their advantage is that the cross girder span and construction depth are reduced compared to the full height E Type and spans of up to 26 m can be achieved with a structure depth of about 500 mm.

The logistics of fabrication and transport to site often dictate the physical size of components and spliced connections may be required for construction of very long span main girders. These bridges are usually constructed alongside/near to the existing structure and installed by sliding or transporter methods.

4.3.6 Use of orthotropic deck floors

The use of an orthotropic deck, rather than a composite deck, is an alternative for most of the types discussed above. The floors are usually based on the fairly heavy orthotropic deck used with the standard box girders (Section 4.4) For this form of floor, the Tee stiffeners are usually arranged at 600 mm

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centres and are connected to the main girders with 4 or 6-bolt connections (U-frames are usually arranged at every third cross girder).

The deck plate is usually kept clear of the web between cross girder positions and is not assumed to contribute to the bending strength of the main girders. See details in Section 10. The design of steel deck floors was developed for the standard box girder bridges (see below) and steel decks used with half through plate girders are often based on the standard details for the box girder bridges.

Lightweight orthotropic floors (with longitudinal and transverse stiffening elements) are likely to be susceptible to fatigue damage arising from stress reversal in the continuous components of the floor and are generally not preferred. Network Rail also requires such decks to be subject to a full dynamic analysis for rail speeds in excess of 100 mph.

4.4 Standard box girder bridges

The standard box girder bridge design was developed by British Rail and Railtrack as an alternative to the E Type deck. The concept aimed to reduce the span of the deck, and therefore its depth, and to simplify erection by crane during possessions. Single track and double track versions are detailed on standard Network Rail drawings[57].

The bridges are half through types, employing trapezoidal steel box girders with inclined inner webs and orthotropic deck units (sometimes referred to as 'battledeck' units). Two versions exist for double track (Figure 4.13). The three girder version uses the same girders as the single track design on the outside of the two tracks and an additional 'six foot' girder with both webs sloping located in the space between the tracks.

The cess walkway and continuous position of safety is provided at the top flange level, rather than at the level of the track. The arrangement of footway is shown in Figure 4.14. It is convenient to accommodate the services beneath the walkway, rather than at track level.

A key feature of the behaviour of the box girder type is that the floor of the bridge is simply supported between the box girders to avoid end fixity effects being generated and transferred to the box girder web. The virtual pinned connection is achieved through a special type of shear plate detail that allows relative rotation between the ends of the cross girders and the box girder. As well as the very great benefit to fatigue performance (see comment in Section 8.6.3), the pinned connection offers the benefit that highly skew arrangements can be achieved without the complications in trimmer details inherent in other forms of construction. The box girders have the torsional and distortional stiffness necessary to carry the eccentric loading applied through the shear plate and the box girders are supported on linear rocker bearings beneath the inner sloping web.

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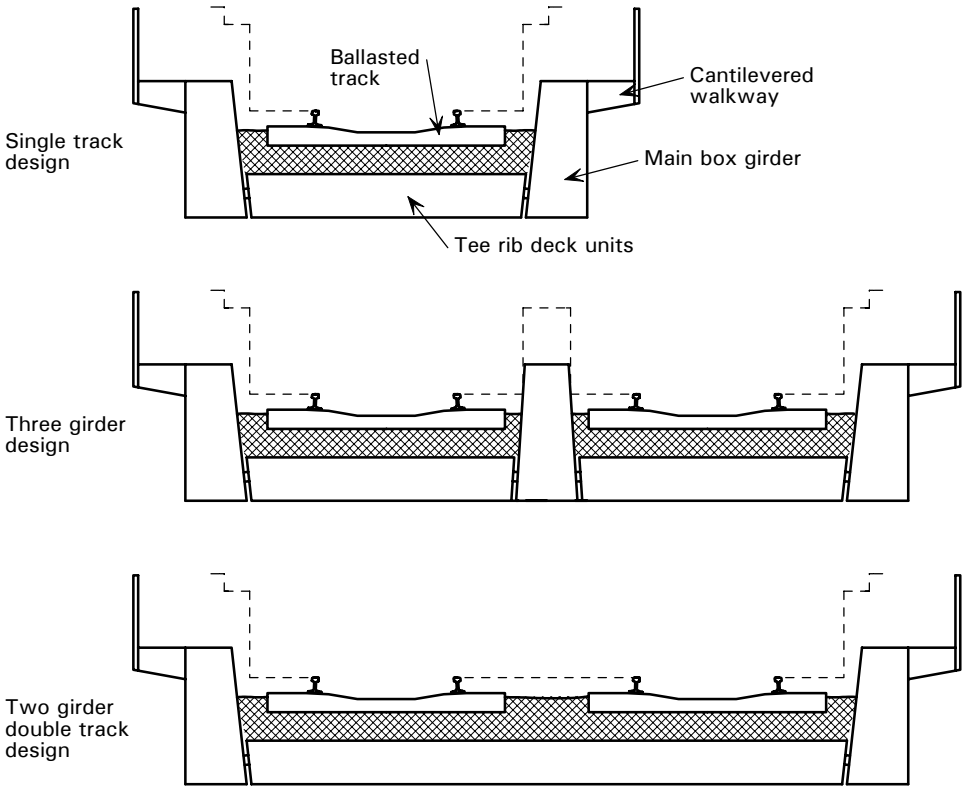

Figure 4.13 *Simplified cross sections of the three design ranges covered by the 'standard box girder bridge' design*

Structure depths vary from 260 mm for the single-track and three-girder designs, to over 350 mm for the wide two-girder design.

Up to 27 m span, the main girders of all three types can be fitted to platform gauge clearances and walkways are provided at top flange level by cantilevering from the main girders (Figure 4.14).

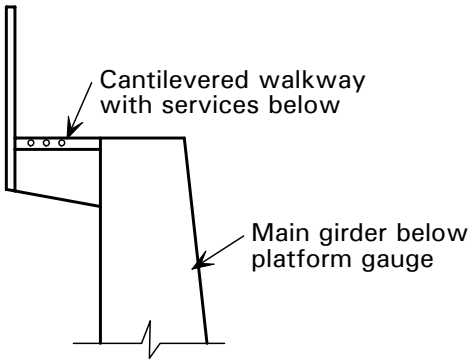

Figure 4.14 *Part section showing box girder with cantilever walkway*

The two-girder double track design can be used above 27 m span, up to a maximum span of 39 m. Within this range, the height of the main girders exceeds platform gauge and the deck width is increased to provide sufficient clearance between the girders for a walkway/continuous position of safety at ballast level, in a similar manner to the D/E Type decks.

The standard drawings (1996 issue) include full details of the components within the standard decks and substructure loadings such that numerical analysis is not

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required. The drawings cater for the span ranges mentioned above, various deck widths and skews of up to 55 degrees.

The box girders form confined spaces and, for smaller spans (which need smaller boxes), other structural configurations should be considered wherever possible. Entry to the box girders is obtained at the girder ends and the standard drawings contain designs for inspection units, which either bolt onto the ends of the girders, or are integral with them, cantilevering back from the girder ends. Two versions are included for side entry from the external elevation and top entry from track level. It is also acceptable to design an independent access chamber/pit butting up to the girder ends, as part of the substructure design. The six foot girders of the standard three girder design have particularly narrow inspection units at the lower end of the span range and consideration should be given to increasing the six foot dimension (track spacing), or changing to an alternative form of construction. Weathering steel should be considered so that the operations of applying and maintaining a protective coating within a confined space can be eliminated (although a coating is applied to the lower half of the inside of the box before it is closed during fabrication).

Box girder bridges of larger spans, increased widths and continuous multi-span girders have also been designed to suit specific sites. Such designs are not covered by the range of the standard drawings and must be fully analysed. Consideration should be given to the provision of additional access hatches within the span of larger span and continuous girders for inspection requirements.

4.5 Slab-on-beam composite girder bridges

Where construction depth is not critical, steel plate girders or rolled sections spanning longitudinally, with a reinforced concrete deck acting compositely with the steel sections, may be employed (i.e. a deck similar to that employed for highway bridges).

This form of construction can be used for spans from 8 m to over 50 m, in single span or continuous span construction. Structure depth to span proportions usually lie in the range 12 to 15 for simple spans.

The girders are supported on structural bearings and the deck slab can be designed to support parapets and to provide a robust kerb.

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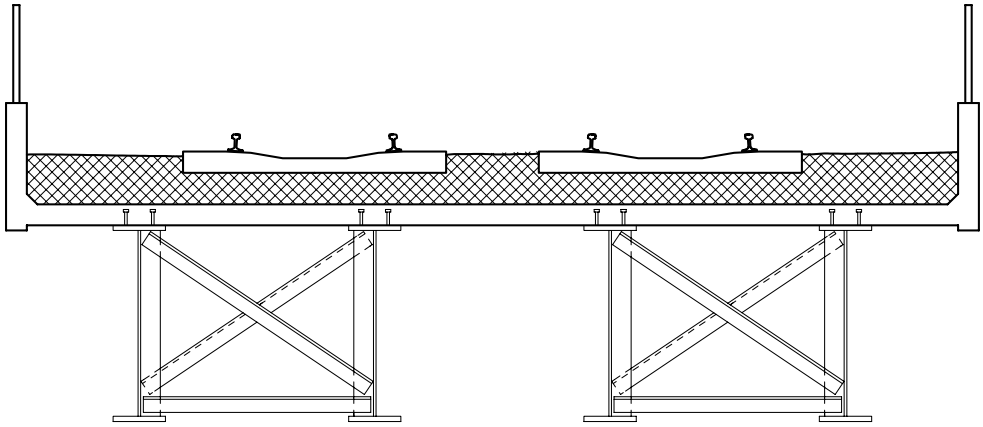

Figure 4.15 *Cross-section of a composite superstructure*

For highway bridges, deck slabs are often cast in situ but this is impractical for reconstruction during typical railway possessions. To overcome this, superstructures have been constructed in advance and installed by sliding, or transporter methods (see Section 3.5). Precast decks have also been used, with the shear connection to the main girder completed in situ.

Composite construction with multiple I-section main girders is also used for railway bridges. See Figure 4.16.

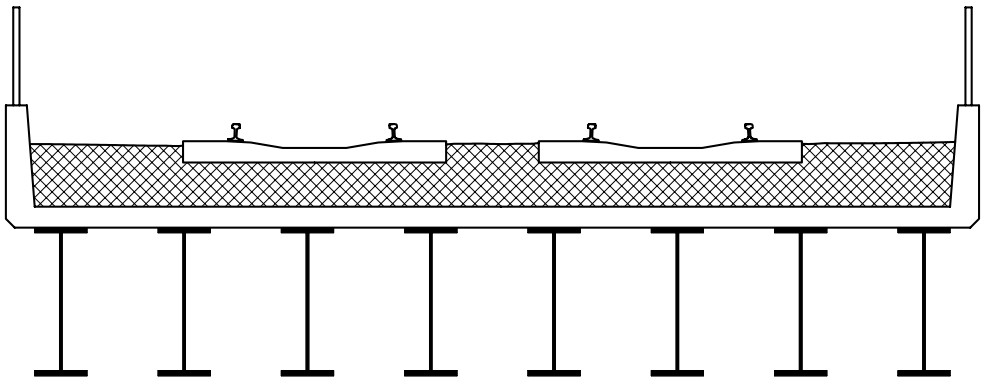

Figure 4.16 *Composite deck-type bridge with multiple girders*

4.6 Truss girder bridges

The design of truss railway bridges is beyond the scope of this publication, but the following comments are given, for general information.

Half through and through superstructures with truss girders have been employed for large span structures and a few new and replacement bridges of this type are still being built. In most cases, a truss configuration is the only viable option for a large span. Historically, a wide range of truss forms have been used but today only Warren Truss, Modified Warren Truss or Bowstring Truss forms are normally considered.

For spans under about 80 m, half through construction is likely to be used and U-frame action will need to be developed to restrain the top chords. The details of the connection of cross girders to bottom chord nodes are inevitably complex and very careful consideration needs to be given to design for fatigue endurance. For larger spans, through construction will be used and the top chords are stabilised laterally by bracing in the plane of the chords.

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Usually, cross girders are provided at positions of the nodes on the bottom chords and the floor spans longitudinally between these cross girders. Historically, stringers or waybeams were provided, but this open form of construction has disadvantages in having many intricate details, which unless very carefully designed give rise to fatigue problems, difficulties in maintenance and openness to noise transmission. Currently there is a strong preference for a reinforced concrete slab extending between the trusses whilst spanning longitudinally between cross girders. A robust kerb is provided inside the line of the trusses to contain derailed vehicles and provide protection to the truss girders. In some cases, where weight or structural depth is at a premium, orthotropic decks may be used.

Where through construction with overhead bracing is used, consideration of vertical clearances must be made to allow clearances to all vehicles identified for use on the route. In most cases, clearance to overhead electrification equipment is also required, even if the line is not currently electrified.

Signal sighting may be an issue with deep trusses and may result in wider spacing of the trusses than is needed simply for clearances.

Because of their physical size, truss girder bridges are normally assembled on site from a series of individual members/subassemblies and installed by sliding during possessions. Crane and transporter methods have been used, but are rare because of the weight involved and time constraints.

4.7 Direct fastening construction

The forms of construction described above all cater for conventional ballasted track. Where the depth available for construction is severely limited by headroom and track level constraints, dispensations to use lower depths of ballast are often necessary and, in some cases, the rails have to be directly fastened to the structure. Direct fastening versions of all of the above solutions, with the exception of solid steel slabs are available.

Direct fastening is normally achieved by either mechanical fixings, or embedding rails within an elastomer in a trough attached to, or cast into the deck.

Direct fastening is generally avoided, if at all possible, because of the onerous maintenance requirements.

4.7.1 Direct fastening systems

There are two basic systems:

- Mechanical fixings between the deck and rails
- Embedding the rails within an elastomeric compound in grooves formed within the deck.

Mechanical fixings can comprise baseplates, or clip housings that are cast/fixed using bolts/studs into the deck. Special adjustable sprung baseplates with elastomeric pads beneath and spring-loaded fixings are often used. They have the advantage of reducing the stiffness of the track and can also reduce noise. By eliminating the ballast and the sleepers, construction depth is reduced by approximately 450 mm compared to conventional track (with 300 mm of ballast beneath sleepers).

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A further reduction in construction depth can be achieved where embedded rails are used. For this track form, the rails are contained within grooves, which are usually created using steel trough section rail bearers, or cast into concrete members. They are commonly used for paved tramways. The rails are aligned within the grooves using wedges of hardened elastomer and then embedded by pouring liquid elastomer into the trough. The resultant track is more flexible than mechanically fixed track and is generally less prone to component failure. The construction depth savings at least equal those with mechanical fixings and if the grooves can be incorporated into a longitudinal spanning deck, a further saving of approximately 150 mm is achievable.

4.7.2 Transition arrangements

Normal ballasted track is quite flexible and deforms within acceptable limits under traffic. Consequently, there is a severe change in flexibility at the start and end of a length of directly fastened track. The horizontal and vertical alignment of the track at the approach to such a structure tends to become uneven under traffic and difficult to maintain. The mechanical/other fixings at the start and end of the directly fastened track are also subject to alternating loadings, as the adjacent ballasted track deflects under axle loads.

One solution to the transition problem between ballasted and directly fastened track is to provide running on slabs. A running on slab, sometimes referred to as a transition slab, is usually a reinforced concrete slab, butting up to the end of the bridge deck, supported at one end on the bridge abutment. Guidance should be sought from the railway authority about transition arrangements.

On London Underground, bridge running-on slabs are deemed unnecessary, because axle weights and train speeds are significantly lower.

Ballast gluing has also been used to create a transition within ballasted track on the approaches to existing bridges, by injecting layers of ballast with a resin compound.

4.7.3 Waybeams

Waybeams are longitudinal members to which rails are directly fastened. They might be used for some light rail, short span, low speed situations or for temporary works but are no longer used for mainline railways. For small spans, waybeams can be used to span longitudinally between abutments and they can also be used spanning between cross girders in trusses, where open construction is acceptable.

Steel plate decking is often used in conjunction with waybeams (either side of the track); the plate must then be designed to cater for derailment.

4.8 Integral construction

Historically, there have been different ways in which bridge movements were catered for (if at all). For shorter spans, the girders were usually simply seated on the abutments, with no specific provision for either movement or restraint. Medium to long spans often had bearings that were intended to allow thermal movements to occur but in practice they often seized up over time and unintended ‘integral’ behaviour occurred. Currently, there is a significant interest in ‘integral bridge construction’ for highway bridges, with the intent of eliminating all movement joints and, where possible, all bearings as well. The

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superstructure and substructure act together in response to loading and thermal expansion/contraction.

Fully integral construction, where there is rotational continuity between substructure and superstructure, is unlikely to be appropriate for replacing existing railway bridges, because of the difficulties in making the necessary connections within possessions, the likelihood of critical details being hidden beneath the track, the maintenance liabilities and the onerous fatigue effects that such connections would attract.

However, in some circumstances a form of 'pinned integral construction' may be appropriate, particularly where it is beneficial for the deck structure to act as a prop between existing abutments that would otherwise have inadequate stability. This may be the case for some replacement bridges.

Some general guidance on integral construction may be found in an SCI publication[43], but if integral construction is to be considered for a railway bridge, careful consideration must be given to the particular features and requirements for railways and early consultation with the railway authority is recommended.

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5 DESIGN STANDARDS

5.1 HMRI principles and guidance

Railway Safety Principles and Guidance[49] published by HM Railway Inspectorate (HMRI) gives general recommendations (Part 1) and detailed guidance relevant to bridges (Part 2 Section A, *Guidance on the Infrastructure)*. Strictly, this guidance is not mandatory. However, HMRI is required by law (under the *Railways and Other Transport Systems (Approval of Works, Plant and Equipment) Regulations*[32]) to approve most new railway works, including bridges, and approval is unlikely to be given to departures from the guidance without good reason. Many of the HMRI Guidance provisions, moreover, are repeated in Network Rail standards or incorporated by reference into them.

For bridgeworks within stations, reference to the *Fire Precautions (Sub-surface Stations) Regulations*[33], Section 12, is required, to determine whether the structure requires an enhanced fire rating. London Underground Engineering Standard E1045[21] lists the stations that must comply with these regulations.

5.2 Railway standards

For UK mainline railways, there are two principal classes of document related to design and construction: Railway Group Standards and Network Rail Company Standards. London Underground has its own set of Standards.

5.2.1 Railway Group Standards

Railway Group Standards are published by the Railway Safety and Standards Board (formerly Railway Safety). The organization is a stand-alone company owned by Network Rail, train operating companies and other interested parties. The Standards are intended to set out minimum mandatory provisions for ensuring safety on Network Rail infrastructure and they impose requirements on train operators and station operators as well as on Network Rail.

In addition to mandatory standards, the Railway Safety and Standards Board also publishes certain Railway Approved Codes of Practice. These contain recommendations which are non-mandatory but which are deemed to constitute "suitable and sufficient means of meeting the mandatory requirements of a Railway Group Standard".

5.2.2 Network Rail Company Standards

Network Rail publishes its own Company Standards (currently many of these are still in the name of the former Railtrack), which respond to Railway Group Standards by adding to and/or setting out in greater detail how the requirements are to be met. The Company Standards are divided into Procedures (generally management processes) and Specifications (generally more detailed technical requirements).

Network Rail also publishes a number of non-mandatory Company Codes of Practice (again, many of these are currently still in the name of Railtrack).

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5.2.3 London Underground Standards

London Underground's Chief Engineer's Directorate is responsible for the preparation and publishing of Engineering Standards.

If standard clearances or any other Engineering Standard cannot be complied with, a concession to Standards must be obtained using the process described in Engineering Procedure P2006[28].

5.3 Technical approval

5.3.1 Network Rail

Network Rail operates a technical approval process for bridges and other civil engineering work, as set out in Company Specification RT/CE/S/003 *Technical approval of design, construction and maintenance of civil engineering infrastructure*[14]. As far as bridge design is concerned, Network Rail's process is similar to (though not identical with) the Highways Agency's technical approval procedures, with an Approval in Principle (AIP) submission - Form A - and a Certificate of Design and Checking - Form B.

The AIP submission is a powerful control on design. The designer is required to list on Form A the standards proposed to be used and any proposed departures. A description of the proposed work is also required, normally accompanied by appropriate drawings. The amount of detail called for at this stage varies from job to job; as a general rule, the more unusual the design the more information will need to be provided. Form A then has to be signed in acceptance on behalf of Network Rail, with or without comments on the proposals prior to detailed design.

5.3.2 London Underground

London Underground operate a similar process for technical approval, as set out in Engineering Standard E1008[20].

5.4 Particular railway standards

5.4.1 Railway Group Standards GC/RT5110 and GC/RT5112

GC/RT5110[6] *Design requirements for structures* sets out general requirements relating to design of all structures. Apart from some provisions concerning bridge parapets over electrified railways, the requirements are all fairly "high level", i.e. they specify principles to be followed rather than giving detailed or quantified rules.

GC/RT5112[7] *Loading requirements for the design of bridges* is more prescriptive. It specifies that railway bridges "shall generally be designed for full RU type loading as specified in BD 37/88 Loads for Highway Bridges" although in exceptional cases a lighter loading may be permitted with suitable justification. Continuous bridges must also be checked for the effects of "load model SW/0" (defined in a diagram) applied to any one or two tracks. An appendix lists the loads and load effects to be considered (including primary and secondary traffic loads, superimposed loads, loads due to the environment and accidental loads) but does not give values.

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Notwithstanding the document's title, GC/RT5112 also gives requirements for the deformation of railway bridges under traffic, by reference to a leaflet 776-3R[35], published by the UIC (Union International des Chemins de Fer).

5.4.2 Railway Approved Code of Practice GC/RC5510

GC/RC5510[5] *Recommendations for the design of bridges* gives a set of recommendations for the design of bridges and supports the requirements of GC/RT5110 and GC/RT5112. Although strictly the recommendations are not mandatory, designers would be wise to follow them unless they have good reason not to or unless they are specifically instructed not to by Network Rail. Any departures from the recommendations would need to be identified and justified in the relevant AIP submission.

Some of the recommendations are general or "high level" but many are specific and quantitative. They cover loading, structural design and non-structural matters (for example clearances, and provision of lineside walkways). In many cases, they refer to other documents such as Railway Group Standards, British Standards, DMRB Standards (see Section 5.6) and UIC Leaflets. They cover train speeds up to 125 mph (200 km/h) but above this speed only in very general terms, such as the need to seek specialist advice.

Structural design recommendations are generally based on BS 5400. There are a few modifications recommended to Part 4 and to Part 5 (in the form of the Yellow Document, see Section 5.6), and to BD 37/88. The current issue of GC/RC5510 was published before the latest revision of Part 3 but it is implied that the revised Standard should now be used. Use of the latest revision of Part 3 is particularly important for designing girders restrained by U-frames and for design of bearing stiffeners, where the new rules are significantly different.

5.4.3 Network Rail Company Standard for bridge design

Network Rail is currently preparing a Company Standard covering the design of bridges. The current documents most relevant to designers are those published by Railway Safety and Standards Board, as given above in 5.4.1 and 5.4.2.

5.4.4 Network Rail Model Clauses

It is arguable whether preparation of contract specifications forms part of design proper, but there is certainly a connection; it is important that the quality of materials and workmanship in a finished bridge structure be consistent with the design assumptions. Network Rail document RT/CE/C/008[12] *Model clauses for specifying civil engineering work* is a comprehensive model framework from which it is recommended that contract specifications be prepared. It is published as a number of separate material-related sections. Section 90 covers steelwork for bridges and is intended to be consistent with design in accordance with GC/RC5510. Section 100 covers bearings (other than steel), Sections 110 to 115 cover waterproofing and Section 80 covers structural concrete

5.5 BS 5400

The design and construction of bridges in the UK is covered by British Standard BS 5400: *Steel, concrete and composite bridges*[1]. The document combines codes of practice to cover the design and construction of bridges and specifications for the loads, materials and workmanship. It is based on the

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principles of limit state design. It comprises the following Parts:

Part 1	General statement
Part 2	Specification for loads
Part 3	Code of practice for design of steel bridges
Part 4	Code of practice for design of concrete bridges
Part 5	Code of practice for design of composite bridges
Part 6	Specification for materials and workmanship, steel
Part 7	Specification for materials and workmanship, concrete, reinforcement and prestressing tendons
Part 8	Recommendations for materials and workmanship, concrete, reinforcement and prestressing tendons
Part 9	Bridge bearings
Part 10	Code of practice for fatigue

The general principles of the limit state design approach are given in Part 1; it states that two limit states are adopted in BS 5400, the ultimate limit state (ULS) and the serviceability limit state (SLS).

Part 2 specifies loads that are to be taken into account in the design. Parts 3, 4, 5 and 10 are Codes, which are manuals of good practice for the design of bridges. Implicit in the Codes is the assumption that workmanship and materials will be in accordance with the Specifications of Parts 6 and 7. These two Parts are written in a form suitable for incorporation in contract documents. In particular, Part 6 provides a comprehensive specification for the various forms of steel (plates, sections, bolts, welds, etc.) and the quality of workmanship employed in fabrication and erection.

In Part 3 (as amended in 2000), reference is made to a number of product standards for the steel material, the most commonly recognised of which is BS EN 10025[2]. Part 6 refers to these and other supporting standards for materials, workmanship, inspection and testing, etc. See further comment in Section 7.5.

5.5.1 Interrelation of Parts 3, 4 and 5 of BS 5400

Part 5 of BS 5400 deals with the design of composite bridges, but provides detailed requirements only for the interaction between steel and composite elements. Design of the separate elements is referred to Parts 3 and 4 of the Standard.

5.5.2 Revised versions of Parts 2 and 5 of BS 5400

Since Parts 2 and 5 of BS 5400 were published by BSI, there have been technical changes to those documents that have been implemented not through the revision of the documents but by issue of new documents by the Highways Agency (those documents are now part of the *Design Manual for Roads and Bridges* (DMRB), see below). In both cases, the revised documents are applicable to railway bridges as well as to highway bridges.

At the time of writing, it is expected that BSI will be reissuing Parts 2 and 5, updated with the relevant technical revisions in the DMRB documents.

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5.6 Design Manual for Roads and Bridges

The Design Manual for Roads and Bridges (DMRB) comprises a collection of Standards and Advice Notes issued by the 'Overseeing Organisations' responsible for highways in the UK. The documents give guidance to the designer and provide interpretation and application of BS 5400. The documents also correct typographical errors in the Standard and amend it where considered appropriate. Most notable in the latter category are documents BD 37 and BD 16.

BD 37/88 was issued in 1988 and made significant revisions to the highway bridge loading; the document included a complete re-presentation of BS 5400-2, with amendments incorporated. The document is referred to in current railway standards (notably GC/RT5112 and GC/RC5510) rather than the BSI document. BD 37 was reissued in 2001 (BD 37/01), including updates to the wind loading and some small revisions to railway bridge loading. It is expected that the railway standards will be revised to refer to the latest version of BD 37.

BD 16/82 implements and makes extensive changes to BS 5400-5; the changes were so extensive that an unofficial compilation of BS 5400-5 and BD 16/82 (sometimes referred to as the "Yellow Document") was produced by the Highways Agency. GC/RC5510 recommends the use of this Yellow Document, rather than the BSI document. At the time of writing (March 2004), BD 16/82 and the Yellow Document are still current.

There is no general requirement for designers of railway bridges to use or comply with other documents in the DMRB, although a few of those documents are specifically referred to. It is recognised, however, that DMRB standards often represent industry best practice and they may generally be used for guidance provided that they do not conflict with any particular requirements or recommendations applicable to railways.

5.7 Matters not covered by standards

Ideally, from the point of view of designers, everything required to design a bridge would be covered by published standards. Regrettably, however, life is not as neat as this. For various reasons (e.g. changes in commercial policy, technological development, increases in speed of trains, recent problems or incidents or accidents), the railway authority may impose requirements that are not covered in any published standard.

Designers would therefore be well advised to maintain close liaison with Network Rail or the appropriate London Underground infrastructure company during all stages of the design process, especially the early stages leading up to formal AIP submission. Particular matters in respect of which Network Rail is known to have current requirements not properly covered in standards include the following:

- Dynamic effects due to passenger trains at speeds above 125 mph (200 km/h) or heavy freight trains at speeds above 60 mph (100 km/h).
- Effects on local deck elements of direct-fastened bridges due to heavy instantaneous wheel loading caused by wheel flats.
- Deck-end uplift of bridges under traffic where the deck overhangs the bearings appreciably.

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- Design of bridge strike impact protection beams for heavily-skewed bridges.

Design requirements for bridges on high speed lines must also comply with the Railways (Interoperability) (High Speed) Regulations[34] and its associated technical specification for interoperability. Such bridges are outside the scope of this publication. Advice should be sought from the railway authority.

5.8 Development of Standards

Network Rail and London Underground Standards are subject to a continual process of review and updating to take account of changing circumstances. The onus is on the designer to ensure that reference is made to the latest issues of the relevant documents.

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6 DESIGN PROCEDURES

In this Section, some references are made to specific clauses in BS 5400-3 and in the Yellow Document version of BS 5400-5. For brevity, the clause references are given in the form 3/1.2.3 and 5/1.2.3, respectively.*

6.1 General

The design process encompasses the whole range of activities from initial selection of the basic form of the bridge, through the sizing of individual elements to detailed numerical checking against recognised criteria. Earlier Sections of this publication have set out functional requirements, practical considerations for construction and the forms of construction commonly used for railway bridges. This Section discusses the influence of these requirements and considerations on the design process and indicates how the key aspects of the various design standards referred to in Section 5 are taken into account during design. Further aspects of design, relating to specific details, are discussed in Section 10.

Adequacy of structural performance is measured against three basic criteria: strength, fatigue endurance and deformation performance. For railway bridges, all three criteria can have a strong influence on design, even for very modest spans. It is therefore important for railway bridge design that all three are considered from the earliest stages of design.

Generally, railway bridge design in the UK will be in accordance with BS 5400. The various documents referred to in Section 5 are essentially either implementation directives or supplementary documentation. The design rules for structural elements are independent of the bridge type, so general guidance on application of code rules for steel and composite highway bridges is applicable to railway bridges. However, the forms of construction and design loadings are different, so different emphasis is needed. Some of the guidance below has been extracted from SCI publication P289[50], but the extracts are limited to those aspects most likely to be relevant to railway bridges; if more detailed guidance is needed (such as on the design of beams with longitudinal stiffeners) reference should be made to that publication.

The first step in the design process is to identify and understand the client and project requirements (this includes obtaining the relevant standards, etc.). A key aspect of those requirements is the bridge loading; guidance on loading is given in Section 6.3. Based on the requirements, it will usually be possible to select an initial form of construction and make an approximate sizing of the principal elements; guidance is given in Section 6.4.

Before setting out on verification of the adequacy of the initial selection, global analysis is needed to determine the load effects (forces, moment and displacements); see Section 6.5. Guidance on verifying adequacy is given in Sections 7 to 9. Bearing selection/design and interface with the supporting substructures is also very important; see Sections 7.8 and 7.7 respectively.

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6.2 Design basis

BS 5400 adopts a limit state approach, in which 'nominal loads' are multiplied by 'load factors' (γ_{fL}) to derive 'design loads'. These design loads are then applied to the structure in isolation or in combination to determine the most adverse internal forces (bending moments, shears etc.), which are in turn utilised to calculate resultant 'design load effects'. The design resistance is in turn based on 'characteristic' material properties, reduced slightly by the application of another partial factor on material strength (γ_m). The criterion for structural adequacy is expressed as:

$R^* \geq S^*$

i.e. the design resistance R^* shall be at least equal to the design load effects S^* – see Clauses 2.3 and 5 of Part 1.

In this verification process, in addition to the partial factors γ_m and γ_{fL}, there is a third factor γ_{f3} to be applied. Unfortunately, there is an inconsistency between Parts 3, 4 and 5 of BS 5400 in applying that factor. In Part 3 (steel bridges), the calculation of design resistance involves division by γ_{f3} whereas in Part 4 (concrete bridges) the calculation of design load effects involves multiplication by γ_{f3}. It is not important whether γ_{f3} is applied as a divisor on the strength side or as a multiplier on the loading side but, clearly, care must be taken when considering a composite structure that γ_{f3} is neither omitted nor applied on both sides. It is recommended here that γ_{f3} always be applied as a divisor on the strength side.

Attention is also drawn to the different treatments of the partial factor γ_m. Part 3 gives values for γ_m that are to be applied in various circumstances to expressions for design strengths (resistances); the factor is explicitly applied in the calculation. In Part 4, γ_m is often implicitly included in expressions for design strength (such as ULS moment resistance of a slab).

The following notes offer a fuller explanation of the key terms used in the above discussion:

Nominal loads (Q_k)	are unfactored loads, they are the loads that would be applied to a structure in a perfect situation, i.e. in the case where the dimensions, densities and/or applied loads are known precisely.
Load factors (γ_{fL})	are the partial factors applied to the nominal loads to cater for uncertainty in the value of the applied load (i.e. a vehicle/train may be overloaded etc.), and for the probability of coincident loading events occurring simultaneously (load combination).
Load combination	A combination of types of load that might reasonably coexist. It is reasonable to design for the worst credible combinations of loads that might occur, rather than assuming that the most adverse of each type of load occurs simultaneously. (For example, it is statistically unlikely that an overloaded train runs at full speed on the coldest day of the year in an extreme wind.)

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Design loads (Q^*)	are the products of the nominal loads and their load factors.
Design load effects (S^*)	are the moments, forces, stresses and displacements resulting from the response of the structure/ element to the design loads
Characteristic strength	A value of material strength that can 'reasonably' be relied upon to be achieved when the structure is constructed. Usually the guaranteed or specified minimum value is used.
γ_m	A partial factor to allow for uncertainty in achieving the characteristic strength
γ_{f3}	A partial factor to allow for other uncertainties – for example in the accuracy of model analysis.

6.3 Loading

The loading to be used for railway bridge design is essentially that specified in BS 5400-2. However, as mentioned in Sections 5.5 and 5.6, that document was modified by BD 37/88 and more recently by the updated BD 37/01[40]. For mainline railways, document GC/RT5112 specifies that BD 37/88 shall be used and document GC/RC5510 gives recommendations that further modify BD 37/88. References that are made to 'Part 2*' in this Section refer equally to BS 5400-2 and to both issues of BD 37 unless otherwise noted.

The loading specification in these documents gives values for all live loads and defines the basis of deriving dead loads. The specification includes a summary table of the various load factors that are to be applied in the various load combinations for each of the load types. An extract from that table, giving only the factors relevant to railway bridges and modified according to GC/RC5510, is included here as Table 6.1.

The principal types of loading to be considered are:

- Dead load.
- Superimposed dead loads.
- Railway Live Loading.
- Accidental loading.

Each of these types is discussed below.

Further specification clauses relate to loads due to wind and temperature effects. In any design, all loadings should be considered, although for many railway bridges, the loading combinations with temperature and wind loads do not govern.

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Table 6.1 *Summary of load factors for railway loading*

Clause number	Load		Limit state	γ_{fL} to be considered in combination				
				1	2	3	4	5
5.1	Dead:	Steel	ULS	1.10	1.10	1.10	1.10	1.10
			SLS	1.00	1.00	1.00	1.00	1.00
		Concrete	ULS	1.20	1.20	1.20	1.20	1.20
			SLS	1.00	1.00	1.00	1.00	1.00
5.2	Superimposed dead:	Ballast	ULS	1.75	1.75	1.75	1.75	1.75
			SLS	1.20	1.20	1.20	1.20	1.20
		other loads*	ULS	1.20	1.20	1.20	1.20	1.20
			SLS	1.00	1.00	1.00	1.00	1.00
5.1.2.2 & 5.2.2.2	Reduced load factor for dead and superimposed dead load where this has a more severe total effect		ULS	1.00	1.00	1.00	1.00	1.00
5.3	Wind:	during erection	ULS		1.10			
			SLS		1.00			
		with dead plus super-imposed dead load only, and for members primarily resisting wind loads	ULS		1.40			
			SLS		1.00			
		with dead plus super-imposed dead plus other appropriate combination 2 loads	ULS		1.10			
			SLS		1.00			
		relieving effect of wind	ULS		1.00			
			SLS		1.00			
5.4	Temperature:	restraint to movement, except frictional	ULS			1.30		
			SLS			1.00		
		frictional bearing restraint	ULS					1.30
			SLS					1.00
		effect of temperature difference	ULS			1.00		
			SLS			0.80		
5.6	Differential settlement		ULS	1.20	1.20	1.20	1.20	1.20
			SLS	1.00	1.00	1.00	1.00	1.00
5.7	Exceptional loads			to be assessed and agreed between the engineer and the appropriate authority				
5.8	Earth pressure: retained fill and/ or live load	vertical loads	ULS	1.20	1.20	1.20	1.20	1.20
			SLS	1.00	1.00	1.00	1.00	1.00
		non-vertical loads	ULS	1.50	1.50	1.50	1.50	1.50
			SLS	1.00	1.00	1.00	1.00	1.00
		relieving effect	SLS	1.00	1.00	1.00	1.00	1.00
5.9	Erection: temporary loads		ULS		1.15	1.15		
			SLS		1.00	1.00		
6.8	Vehicle collision loads on bridge supports and superstructures:	Effects on all elements excepting elastomeric bearings	ULS				1.50	
		Effects on elastomeric bearings	SLS				1.00	
7	Foot/cycle track bridges:	live load & effects due to parapet load	ULS	1.50	1.25	1.25		
			SLS	1.00	1.00	1.00		
8	Railway bridges:	type RU and RL, and SW/0 primary and secondary live loading	ULS	1.40	1.20	1.20		
			SLS	1.10	1.00	1.00		

ULS: ultimate limit state SLS: serviceability limit state

Shaded values are those given by GC/RC5510

* 'Other loads' includes track and ballast in excess of that between top of sleeper and 300 mm below sleeper (see GC/RC5510)

NOTE. For loads arising from creep and shrinkage, or from welding and lack of fit, see Parts 3, 4 and 5 of BS 5400, as appropriate.

6.3.2 Dead load

The dead load is the weight of the structural elements. It can be calculated from the specified geometry and the densities of the materials. Note that the Part 2* partial factors on dead load for steel and concrete are varied by GC/RC5510.

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6.3.3 Superimposed dead load

Superimposed dead load covers the weight of the waterproofing layer, the ballast, the track and any walkways, parapets, services etc. Again, this can be calculated from the specified geometry and densities of the materials. However, because there may be less control over the extent of the items making up superimposed dead load during the life of the structure, the partial factors are usually larger than those applied to dead load.

For main line railways, the density to be used for ballast is given in GC/RC/5510, which also clarifies the appropriate partial load factors. Ballast depth is usually taken to the top of the sleepers; allowance should be made for increased ballast depth where the deck is sloped for drainage or precambered.

Weights of track

Weights of track components are given in RT/CE/C/025 *Assessment of underbridges*[13]. For weights of London Underground track components, refer to Engineering Standard E3314[23].

To avoid 'double-counting', track weights are often quoted as 'extra over' values (i.e. the weight of the sleepers, rails, etc.) minus the weight of the displaced ballast).

6.3.4 Railway live loading

There are two classes of live load – primary live load and secondary live load. The primary live loads are the weight of the traffic (i.e. vertical loads) and the secondary live loads are horizontal loads that may or may not coexist with the primary loads. However, because railway secondary live loads frequently do coexist, both primary and secondary loads must be considered together in combinations 1, 2 and 3.

For bridges carrying mainline traffic, the applicable nominal loading in known as Type RU; in some cases an alternative loading, known as SW/0 must also be considered. For passenger rapid transit railway systems, the applicable nominal loading in known as Type RL.

For mainline railways, document GC/RT5112 specifies that, in normal circumstances, bridges be designed for RU loading (currently it refers to RU loading as specified in BD 37/88). Additionally, for continuous bridges, load model SW/0 must also be considered.

For London Underground bridges, document E3304[22] specifies that bridges carrying mainline traffic be designed for RU loading and designates to which sections of the system the requirement applies. Elsewhere, the design loading is RL loading, as specified in BD 37/88.

RU and SW/0 load models

Type RU loading is not the weight of a real train but is a simplified model developed by the International Union of Railways (UIC) that, when applied to a simply supported beam, would produce load effects approximately equal to or slightly in excess of those that would be derived if the (static) weight of 'real' trains were applied. The derivation of the RU load model is given in UIC document 776-1R[36].

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Type SW/0 loading is a special loading that only needs to be applied to continuous bridges, where the RU model does not give the worst loading effects at the intermediate supports. The derivation of the SW/0 loading is also given in UIC document 776-1R. The load model should be curtailed if it produces a more onerous effect.

Details of RU and SW/0 loading are given graphically in Figure 6.1 and Figure 6.2

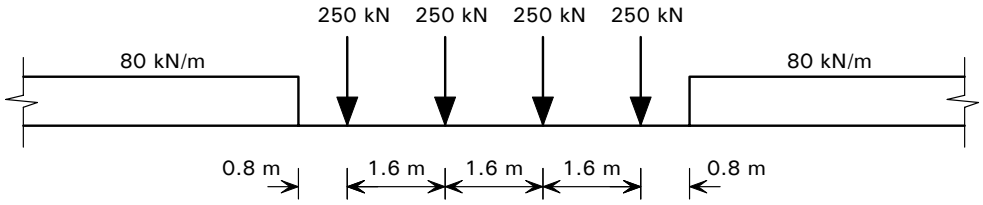

Figure 6.1 *Type RU loading (UIC loading 71)*

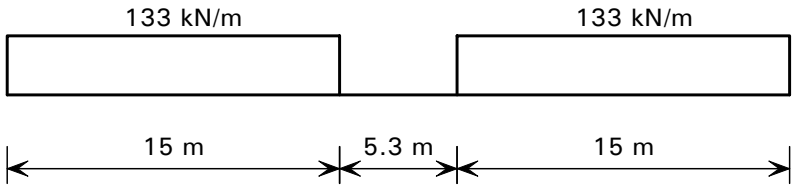

Figure 6.2 *Type SW/0 loading (for continuous spans only)*

RL Loading

Type RL loading is also a simplified model that is intended to produce load effects equivalent to those of 'real' mass-transit and light railway trains. The loading model is shown in Figure 6.3. For deck elements, Part 2* states that an alternative of two concentrated loads also needs to be considered.

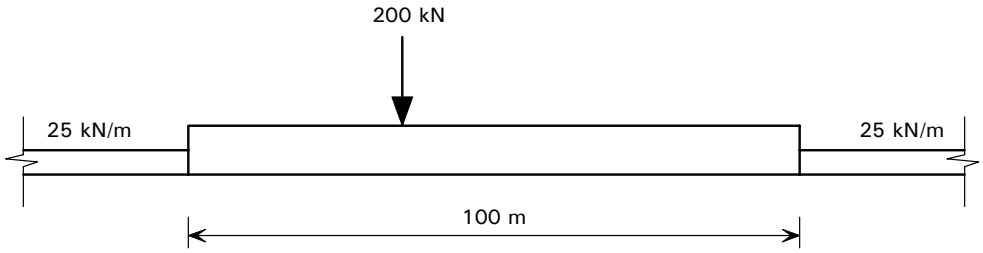

Figure 6.3 *Type RL loading*

Dynamic effects of vertical loading

RU, RL and SW/0 loads represent the static weight of the traffic. To take account of the magnification in load effects that may develop under the approach, passage and departure of a train travelling at speeds up to 200 km/h, a 'dynamic factor' is applied. (See comment in Section 2.3 on the aspects of dynamic behaviour that are considered – aspects a, b, c and f are taken into account by the dynamic factor.)

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For RU loading, the value of the dynamic factor is given in an expression that includes the length of the 'influence line'† for the element being considered, which is taken to represent the natural frequency of the element. Different factors are given for moment effects and shear effects, because the magnification of bending moments is greater than that of shear forces.

For RL loading, two simple factors are given, one for ballasted track, one for unballasted track, both irrespective of span, but the factor does not allow for lurching (aspect f in Section 2.3), which is separately allowed for by applying 56% of the load to one rail and 44% to the other rail.

For train operating speeds in excess of 200 km/h (125 mph) the dynamic factors are not applicable and special consideration is needed. Consult the railway authority for appropriate requirements.

Secondary live loads

Secondary live loads are those caused by the change in speed/direction of the train that causes primary loading. The three types of secondary load are: centrifugal loading, nosing and longitudinal loading (braking/traction).

Centrifugal Loading

The 'centrifugal' load due to a mass travelling around a curve at speed is easily calculated. In Part 2*, the speed applicable is 10 km/h higher than the highest speed expected. This gives the formula:

$$F_c = \frac{P(v_t + 10)^2 \times f}{127r}$$

Where P is the static load, v_t is the train speed (km/h) and r is the radius of curvature (m). The parameter f is a reduction factor that recognizes that trains travelling in excess of 120 km/h will be lighter (lower mass) and thus the centrifugal force is less.

Nosing

Lateral oscillation of the train on the track also gives rise to lateral forces. The value of 100 kN given in Part 2* was deduced for this dynamic wheel/rail interaction force from measurements of forces on rails. It should be applied on both straight and curved track.

Longitudinal Loads

Forces due to traction and braking both act along the rails. With continuous welded track, some of the force is transmitted beyond the bridge (see Section 6.5.8). Traction and braking differ in that there are usually only a small number of driving axles, but wheels are braked all along the train. For long loaded lengths, the braking loads are therefore significantly higher than the traction loads.

† The length to be used in deriving the dynamic factor is given in Table 17 of Part 2. An addendum to the Table is given in Appendix I of GC/RC5510 but that Addendum erroneously lists 'concrete slab decks' as elements of battle deck floor; that line of the table should be considered as separate from the definitions for battle deck floors.

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Simplified RU and RL loading

As a further simplification of the RU and RL load models, Appendix D of Part 2* provides tables of equivalent static and dynamic uniformly distributed loads for bending moments and end shear forces for simply supported beams. The tabulated loads are the total loads that, when applied uniformly to a simple span, give exactly the same bending moment and shear force as would RU/RL loading, disposed along the span so as to achieve maximum effect.

Note that the tables apply only to simply supported beams spanning parallel to the direction of the track. For cross girders, such as in a typical half through bridge deck, the RU/RL load model should be applied directly.

Distribution of axle and wheel loads

For the determination of bending moments in transversely spanning elements, the railway loading may be applied along the lines of the rails (usually taken as at 1.5 m centres, see page 1). RU loading should be shared equally between the two rails; RL loading should be shared 56%:44%, as noted above. Guidance on the local effects due to wheel loading is given in Appendix I of GC/RC5510.

6.3.5 Accidental loading

Accidental loading can arise from the derailment of a train or from the impact of a vehicle collision.

Derailed trains

Document GS/RC5510 recommends that 'robust kerbs' be provided to contain the wheels of derailed trains and gives values of the design load for such kerbs. Main girders are deemed to act as robust kerbs provided that they are of sufficient height and are sufficiently restrained laterally.

Clause 8.5 of Part 2* specifies the vertical loading to represent the effects of a derailed train. This clause applies a 'static' load anywhere within 2 m either side of the track (in practice, up to the face of the robust kerb, where this is closer). Note that Clause 7.3.3 of GC/RC5510 says that 19.1 of that document gives both the horizontal loading on the kerb and the vertical loading for derailed trains, but 19.1 only gives the value of the horizontal loading; use the Part 2* value for the vertical loading.

In addition to the loading representing the effects of a derailed train, Clause 4.6.1 of Part 2* also requires a check on the overall stability of the structure in the event of derailment (as a ULS condition). Damage to local elements such as walkways and the parapet is acceptable but the whole structure should not overturn or collapse. (This requirement can dictate the position of main girders in deck type construction or preclude the use of large side cantilevers.)

Impact from road traffic

Bridges liable to accidental impact from road vehicles should be designed to resist the appropriate loads specified in GC/RC5510 Appendix J. That document refers in turn to BD 60/94[42].

For London Underground bridges, refer to BD 37/01 Section 6.8 for impact loads.

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6.4 Initial design

6.4.1 Choice of construction type

The choice of the form of construction will usually be constrained by the available construction depth and the span. The following Tables give some indicative guidance on the construction depths for various forms of construction

Table 6.2 *Construction depths, for spans up to 20 m*

Bridge type	Construction depth (mm) for simple span (m) (See Note 1)				
	3	**8**	**12**	**16**	**20**
Solid steel slabs	850	n/a	n/a	n/a	n/a
Orthotropic deck	950	1100	n/a	n/a	n/a
Filler beam	1000	1200	1400	n/a	n/a
Z types	n/a	1000(2)	1000(2)	1000(2)	n/a
U type	n/a	950	950	950	n/a
B and C types	n/a	1020(2)	1020(2)	1020(2)	n/a
D and E types	n/a	n/a	1225(2)	1225(2)	1225(2)
Standard box girder (single track)	n/a	n/a	960	960	960
Standard box girder (two track)	n/a	n/a	1100	1100	1100
Composite deck-type	n/a	1500	1800	2100	2500

n/a = not applicable

(1) Construction depths include a depth of 700 mm for track, ballast and waterproofing, assumed to be 375 mm, 300 mm and 25 mm respectively.

(2) Plus the thickness of the bottom flanges of the main girder and, where applicable, any clearance between the cross girder soffit and the bottom flange of the main girder.

Table 6.3 *Construction depths, for spans exceeding 20 m*

Bridge type	Construction depth (mm) for simple span (m) (See Note 1)				
	20	**30**	**40**	**50**	**over 50**
D and E types	1225(2)	1300(2)	1300(2)	1300(2)	1300(2)
Truss girders	n/a	n/a	n/a	1300	1300
Standard box girder (single track)	960	n/a	n/a	n/a	n/a
Standard box girder (two track)	1100	1170	1170(3)	1170(3)	1170(3)
Composite deck-type	2500	3000	4000	5000	5000 +

n/a = not applicable

(1) Construction depths include a depth of 700 mm for track, ballast and waterproofing, assumed to be 375 mm, 300 mm and 25 mm respectively.

(2) Plus the thickness of the bottom flanges of the main girder and, where applicable, any clearance between the cross girder soffit and the bottom flange of the main girder.

(3) Standard design covers spans up to 39 m only

Where the ballast depth can be reduced, the construction depths in the above tables can be reduced. See discussion under 'ballast' in Section 2.2.1. Providing a clear gap between the cross girders and the bottom flanges of B, C, D, E and Z types will increase the depth slightly.

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6.4.2 Size of main girders

Up to about 25 m span, fatigue considerations are likely to determine the size of the main girder flanges and thus the sizes will depend on the fatigue class and railway traffic. The appropriate limiting stress range can be easily determined from Part 10 (using the simplified procedure, see Section 8.5) and the bending moment is easily calculated from the equivalent UDL for RU loading including dynamic effects, given in Appendix D of Part 2*.

Above 25 m, strength at ULS is likely to govern. The design bending moment is easily calculated from the equivalent UDL for RU loading, multiplied by the load factor, and flange sizes may be determined on the basis of yield strength in tension and a lower strength in compression (typically between about 70% and 80% of yield for half through bridges, depending on the lateral torsional buckling slenderness that might be expected).

A further consideration with longer relatively shallow main girders on half through type bridges with composite concrete floors is compliance with the crack limits at the slab soffit, under combined global and local effects. This may require greater main girder stiffness to limit global strains. Guidance for standard Z type girders is given in the related Railtrack User Manual. For other half through bridge types an early check on this limit should be made.

6.4.3 Skew superstructures

When the railway crosses the road, river or other railway below at a skew, decisions have to be made at the outset how the support of the superstructure is to be arranged.

In some cases, the bridge supports can be arranged square to the main girders, both at abutments and at intermediate supports, and thus a skew structural configuration is avoided. More commonly, the lines of support are skew to the main girders. There are then decisions to be made about the arrangement of cross girders and the twist effects under live load have to be considered.

The arrangement of cross girders in heavily skewed half through bridges needs particular consideration because some U-frames become ‘L-frames’ and the end trimmers may be better arranged either separately supported or pinned (rather than moment-connected) to the main girders.

In skewed bridges, the twist experienced by rail vehicles, as the bridge deflects under live load, can be significant and the limitations on twist of the track may even determine the size of the main girders, in order to provide sufficient stiffness. Trimmer beams often require intermediate support bearings to reduce deflection. In composite deck type bridges, the use of plan bracing to the bottom flange can create an effective box section, which will increase torsional stiffness significantly and thus reduce twist.

These aspects should be considered at the initial design stage, since they affect the whole performance of the bridge and poor choices at this stage cannot be corrected by later detailing refinements.

Some general guidance of the layout of skew railway bridges is given in GN 1.02[47]. Bridge articulation is discussed in Section 7.8.2.

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6.4.4 Multiple spans

Where there are multiple spans, a key issue is whether spans should be independent or continuous over intermediate supports. Factors affecting this decision may include:

- Structural efficiency (in terms of strength), which is improved when spans are continuous).
- Articulation arrangements (see also section 7.8.2), particularly on longer span bridges, where the cumulative movement on a continuous bridge can be difficult to accommodate and which might also require special arrangements in the track such as breather switches.
- Transmission of horizontal loads. On flexible intermediate piers it may be difficult to design to carry the large horizontal loads from braking and traction on separate spans; on long continuous spans the total load at the fixed abutment may be very large.
- Deflections at intermediate supports (see further comment in Section 9).
- Track/bridge interaction (see Section 6.5.8).
- The possibility of uplift at the end supports, with multiple spans. (Depending on the structural configuration, the high live load to dead load ratio can result in uplift at the end bearings of continuous spans.) Tie-down arrangements to resist uplift under live load are not generally acceptable.
- The ability of the structure and substructures to accommodate differential settlement, especially in areas of mining subsidence. This may be a particular problem when reusing existing foundations.

6.4.5 Fatigue and deformation

Either fatigue or deformation performance may well be the governing criterion for a railway bridge and it is wise to carry out a preliminary evaluation of these effects before commencing on detailed design, even if it is thought that strength is likely to be the governing criteria.

6.4.6 Use of weathering steel

As mentioned in Section 2.6, the use of weathering steel may offer benefits in reducing maintenance requirements. The selection of weathering steel should be considered at the initial design stage, since the choice will influence costs, future maintenance regimes and material thickness requirements.

Weathering steel is particularly worthy of consideration for deck type railway bridges, but less suited to half through bridges, where all surfaces on the track side of the girder and areas where rain water can lie such as the flanges will need to be protected against corrosion.

Although the corrosion rate of weathering steel is much lower than of carbon steel, it cannot be discounted and an allowance for some loss of section over the life of the bridge must be made. See Section 7.1.6.

The need to detail the superstructure and substructure to deal with run-off from weathering steel surfaces, in ways that will avoid unsightly rust staining, should be borne in mind when choosing the structural configuration.

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6.5 Global analysis

A global analysis is required to establish the maximum forces and moments at the critical parts of the bridge, under the variety of possible loading conditions. Local analysis of the deck slab is usually treated separately from the global analysis.

Even for a simply supported span, it is now common practice to use a computer analysis to derive load effects and displacements. Software packages are available over a wide range of sophistication and capability, and the selection of program will usually depend on the designer's in-house computing facilities.

6.5.1 Types of computer model

The basis of most commonly used computer models is the grillage analogy, as described by West[54] and Hambly[48]. In this model, the structure is idealised as a number of longitudinal and transverse beam elements in a single plane, rigidly interconnected at nodes. Transverse beams may be orthogonal or skewed with respect to the longitudinal beams.

For a model of a composite deck-type bridge, each beam element represents either a composite section (e.g. main girder with associated slab) or a width of slab (e.g. a transverse element may represent a width of slab equal to the spacing of the transverse elements).

Grillage models cannot determine U-frame effects in half through bridges. However, I-section main beams (which are at the edges of the deck) have a very low torsional stiffness; this effectively makes the deck simply supported along its edges. The end rotations of the successive cross girders given by the model can then be used to determine load effects in the U-frames (see Section 7.4.1).

A typical grillage model for a skew half through with composite deck is shown in Figure 6.4. The main girders are each represented by longitudinal elements, as mentioned above, and the longitudinal bending of the slab is represented by intermediate longitudinal beam elements; locating these under the lines of the rails simplifies application of loading. For skew bridges, the end trimmer is often simply supported independently of the main beams and this is also shown in the model.

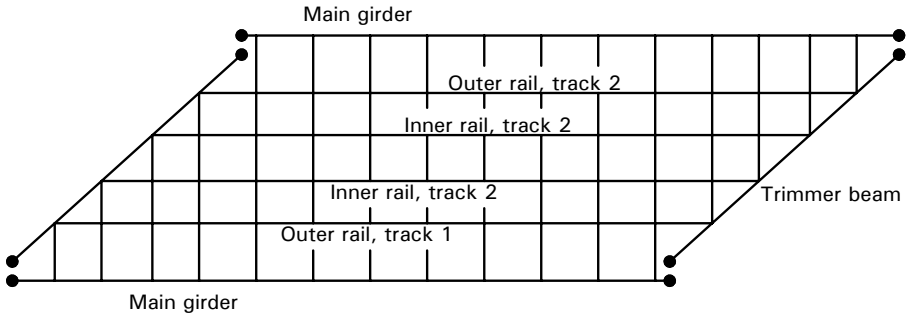

Figure 6.4 *Grillage model for skew half through bridge*

To determine non-buckling U-frame effects directly, linear 3D modelling software can be used as an alternative to the expressions in BS 5400-3. This alternative will be appropriate for heavily skewed bridges in particular. In such an analysis it will be necessary to allow for the flexibility of the connection between cross girders (or deck slab) and the girders; the stiffness of the effective web stiffeners that form part of the U-frame must also be modelled.

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For evaluation of the buckling effects in U-frames, the 3D software must have non-linear or large-displacement analysis capability or the ability to determine eigenvalue solutions for buckling. (Such analysis could be used as an alternative to determining slenderness through the simplified rules for uniform situations that are in Part 3, where agreed by the railway authority.)

The increasing availability of analytical software may lead to wider use of the more sophisticated models, though at present the use of simple grillages is strongly predominant, with the U-frame forces (due to both buckling and non-buckling effects) being calculated in accordance with Part 3 (see Section 7.4.1).

6.5.2 Parameters for global analysis

Half through bridges

The main longitudinal beam elements should coincide with the main girder axes. It is usual to neglect beneficial contribution of the deck so, for determining forces in the main girders, non-composite section properties should be used. However, when considering the SLS crack width checks, the longitudinal shear connection and fatigue checks on the deck and the longitudinal shear connection, an effective width of deck (taking account of shear lag) should be included in determining the beam properties for grillage analysis. (Rules for determining shear lag are given in Part 3 and Part 5.) Because the whole deck is in tension longitudinally, tension stiffening in cracked concrete (for a description of this effect, see Johnson[51]) may be assumed in deriving the effective area of the reinforcement in the composite section . If a 3D finite element model is used, it can take account of the interaction between deck elements and the main beams, provided that suitable element properties are selected.

For the transverse elements, which would normally coincide with the cross girders, the stiffness is the combined bending stiffness of the beams and slab (calculated separately), or of the steel deck plate and cross girder.

The stiffness of the intermediate longitudinal beam elements is that for the slab or for the deck plate, as appropriate.

Torsional stiffness of the slab should be shared between longitudinal and transverse elements, as for deck type bridges (see below). The main girders should be given the St Venant torsional stiffness of the I-section, which will be small and should not generate significant torsional effects.

Grillage models can also be used for half through box girder bridges; the longitudinal elements should be given the torsional properties of the box and dummy stubs may be used to cantilever to the ends of the deck elements, which should be connected with a pin element (or a very short vertical element of very low bending stiffness), if the deck is pin-jointed to the cross girders.

Deck type bridges

It is usual to choose the longitudinal beam elements so that they coincide with the main girders. In a deck type model, the transverse beams do not represent discrete elements and the spacing can be chosen by the designer to facilitate the analysis. Generally, the spacing should not exceed about 1/8 of the span. Uniform node spacing should be chosen in each direction where possible.

Gross section properties should be used in the global analysis (Clauses 3/7.2, 5*/5.1.1 and 5*/6.1.4.1). Section properties for longitudinal beams should

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assume that the fully effective slab is shared between the beams. Long- and short-term load effects should be determined separately, because they should be applied separately to long- and short-term section properties in the stress analysis of sections.

There is no need for the section properties of transverse beam elements to represent transverse bracing or cross girders when the bracing is not continuous (i.e. the bracing is only between pairs of girders).

Section properties for transverse beam elements representing the slab alone should use a width equal to the element spacing. Torsional stiffness of the slab should be divided equally between the transverse and longitudinal beams; use $bt^3/6$ in each direction, where b is the width of slab appropriate to the element concerned.

6.5.3 Choice of cracked or uncracked analysis

Half through bridges

Longitudinal shear connection between the slab and the main girders is usually provided by the cross girder connections. Therefore, the slab will be in tension in simply supported bridges and in midspan regions in continuous bridges. Wherever the concrete tensile stress exceeds $0.1\,f_{cu}$ cracked section properties should be used. See discussion in Section 6.5.2.

Deck type bridges

For the loads applied to a continuous composite structure at ULS, global analysis may be carried out assuming initially that the concrete slab is uncracked over internal supports; up to 10% of the support moments may then be distributed to the span. Alternatively, and more usually, the concrete may be assumed to be cracked for a length of 15% of the span on each side of an internal support (Clause 5*/6.1.4). Similarly, the slab may be assumed initially to be uncracked for SLS and fatigue analysis, but if the concrete stress exceeds $0.1\,f_{cu}$, either a new analysis assuming cracked concrete at the supports is required or the midspan moments must be increased without corresponding reduction in hogging moment (Clause 5*/5.1.1).

Generally, it is advisable for the global analysis for both SLS and ULS to assume from the outset that the concrete is cracked adjacent to internal supports for about 15% of the span. No further redistribution should then be made.

Cracked section properties should include the effective area of the reinforcement over the full width of slab acting with the steel girder. This effective area should take account of the slightly lower modulus of elasticity of the reinforcement.

6.5.4 Analysis of staged construction

Half through bridges

For half through railway bridges, the beneficial effect of structural interaction between the deck slab and the main girders is generally neglected in the design of the main girders, and thus consideration of staged construction is not necessary for their design. However, separate analysis of composite and non-composite models will be needed to determine longitudinal shear at SLS and longitudinal fatigue effects in the slab. With composite cross girders, long- and

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short-term effects will act on different effective sections and staged construction should be considered.

Deck type bridges

It is usual for the deck slab of composite deck type bridges to be concreted in stages, and for the steel girders to be unpropped between supports during this process, although opportunity for propping may arise where a bridge is assembled and concreted off-line. Part of the load is thus carried on the steel beam sections alone, part by the composite sections. A number of separate analyses may be required, one representing each different stage that occurs. This series of analyses will follow the concreting sequence and will take account of the distribution of the weight of wet concrete (Clauses 5*/5.1 and 5*/12.1). These models will be a series of partially composite structures. Where a number of installation methodologies are being considered, each will require a separate series of stage construction models.

6.5.5 Local analysis of the deck slab

Load is transferred from the rails, through ballast, to rectangular patches on the bridge floor, one under each rail/sleeper connection. The size of the loaded area is given by dispersal rules in GC/RT5510, Appendix I. The local effects of this patch loading need to be considered. Local analysis is needed to obtain influence lines for the bending of the slab or deck plate and for deriving the loading history for fatigue calculations. For the local analysis, it may be sufficient to model only part of the bridge deck, using a refined grillage analysis, finite element model or possibly Pucher charts[52]; alternatively, part of the global model can be refined.

For a half through deck with cross girders, all cross girders need to be modelled and a finer mesh of longitudinal elements is needed for evaluating the total bending locally between cross girders – for a steel deck bridge, three lines of beam elements per rail should be sufficient.

For a deck type bridge, with longitudinal main beams, a finer mesh of transverse elements should be used – a longitudinal spacing equal to the slab thickness should be sufficient.

6.5.6 Deformations

The total deflections under unfactored dead and superimposed loads should be calculated to enable the beams to be pre-cambered during fabrication, where this is required (Network Rail do not require spans less than 12 m to be pre-cambered, see GC/RC5510, clause 19.8.4). This information should be produced by the designer and included on the drawings. See GN 4.03[47].

Deformations under live load are needed, to check the deformation performance (see Section 9).

6.5.7 Dynamic response

On high-speed routes (line speed over 200 km/h), there may be interaction between the vibrational response of the bridge and loading frequency generated by the passing axles of the train, above that allowed for in the derivation of the dynamic factors applied to the live loading. Specialist advice should be sought from the railway authority.

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6.5.8 Track/bridge interaction for longitudinal loads

As well as the dynamic interaction between the train and the bridge, there may be track/bridge interaction in resisting longitudinal loads (braking and traction) and expansion/contraction. See guidance in GC/RC5510, clause 12.1, on when consideration of this interaction is needed. For bridges not covered, the document refers to UIC leaflet 774-3R[37] for guidance. Where consideration is needed, advice should be sought from the railway authority. See further comment in Section 9.4.2.

Track/bridge interaction should be considered at the initial design stage, as well as at the detailed design stage.

6.6 Detailed design

The detailed design stage confirms or refines the outline design produced in the initial design stage. It is essentially a checking process, taking the results of global analysis from the application of a complete range of loading conditions to a mathematical model to generate calculated forces, stresses and deflections at critical locations in the structure. These forces, stresses and deflections should be checked to ensure that they comply with the various rules and the adequacy of local detailing should be checked. The detail of the checking process should be sufficiently thorough to enable working drawings to be prepared, in conjunction with a specification for workmanship and materials, and the bridge to be constructed.

Design for strength, fatigue endurance and deformation performance are discussed in Sections 7, 8 and 9 respectively.

6.7 Design of half through bridges

In half through bridges, the degree of structural interaction between the deck and the main girders is a key consideration for designers. Generally, practice has been to ignore the beneficial effects of interaction and to take account of it where it produces adverse effects on the element being considered. Table 6.4 summarises the advice given earlier in Section 6 and also in later Sections.

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Table 6.4 *Allowing for deck/main girder interaction in half through bridges*

ULS effects	
Strength of main girders (longitudinal bending)	For global bending analysis, use bare steel section properties. For stress analysis of beam sections, use bare steel section properties.
Longitudinal shear between deck and girders*	For global bending analysis of simple spans, use bare steel section properties. For global bending analysis of continuous spans, use bare steel section properties in sagging regions and composite properties in hogging regions. May take account of shear lag. For local analysis of slab: Where the slab is in global compression: Use ordinary cracked section properties for bending strength Where the slab is in global tension: Use only reinforcement areas (no concrete) for strength. May enhance reinforcement area to allow for tension stiffening effect.
Axial and bending strength of slab (longitudinally between cross girders)	
SLS effects	
Longitudinal shear between deck and girders	For global bending analysis of simple spans, use bare steel section properties. For global bending analysis of continuous spans, use composite properties For properties in global bending analysis: Take account of shear lag In global sagging regions use cracked section properties but reinforcement area may only allow for tension stiffening effect May consider slab to be uncracked where tensile stress < 0.1 fcu For stress analysis to derive longitudinal shear, use properties as for global bending analysis. For local analysis of slab: Where the slab is in global compression: Use ordinary cracked section properties for crack widths. Where the slab is in global tension: Use only reinforcement areas (no concrete) for crack widths. May enhance reinforcement area to allow for tension stiffening effect.
Crack widths in slab	
Deflection of main girder	Check first with bare steel properties – if the limits are exceeded slightly, check again with composite section properties (reinforcement in cracked regions may be enhanced by tension stiffening), taking account of shear lag.
Fatigue effects	
In main girder	For global bending analysis, use bare steel section properties. For stress analysis use bare steel section properties.
In longitudinal shear connection*	For global bending analysis, use bare steel section properties For section properties in stress analysis of cross sections: Where the slab is in global compression: Use uncracked properties for stresses due to global effects. Use ordinary cracked section properties for stresses due to local bending. Where the slab is in global tension: Use only reinforcement areas (no concrete) for stresses in reinforcement. May enhance reinforcement area to allow for tension stiffening effect. May take account of shear lag.
In slab reinforcement	

* Logically, the analysis for longitudinal shear should be based on the normal composite section, taking account of concrete, to produce a safe result. However, existing bridges dating back to the 1950s, which traditionally only used a standard four-bolt connection to the cross girders, and which were never designed for longitudinal shear, have not exhibited problems in those connections in practice. This is considered to stem from the conservatism of the other design assumptions and the unaccounted benefit of the shear transmission between the concrete and the main girders. Therefore, using the same analysis principles as for the slab design is regarded as acceptable practice, and simplifies design

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7 DESIGN FOR STRENGTH

In this Section, some references are made to specific clauses in BS 5400-3 and to the Yellow Document version of BS 5400-5. For brevity, the clause references are given in the form 3/1.2.3 and 5/1.2.3, respectively.*

While bridges must obviously be designed to be strong enough to carry the maximum loading that is anticipated during the life of the bridge, it must be remembered that a railway bridge is subjected, throughout its life, to very many repeated loadings close to that maximum loading. It is often the case that design for fatigue endurance governs, rather than design for strength. Deformation of the bridge under load is also a primary concern for the safety of the railway traffic and the comfort of passengers and deformation considerations may govern in some situations. See sections 8 and 9 for guidance on design for fatigue endurance and for deformation performance.

7.1 General

The main longitudinal beams must be designed to provide adequate strength in bending and shear to resist the combined effects of global bending, local effects such as compression over bearings, and structural participation with bracing systems. Transverse beams and deck slabs must also be designed to provide adequate strength in bending and shear, although their design is likely to be governed by fatigue.

BS 5400-3 provides two sets of Clauses for evaluating the strength of beams, one for beams without longitudinal stiffeners (3/9.9) and one for beams with longitudinal stiffeners (3/9.10 and 3/9.11). The first evaluates the strength of the beam cross section as a whole, and the second evaluates the strength in terms of its elements (flange panels and web panels, between stiffeners). In the present publication, guidance is restricted to parallel flange beams without longitudinal stiffeners (beams of variable depth are treated as longitudinally stiffened beams and are therefore also not covered here).

Additionally, for beams without longitudinal stiffeners, Part 3 (and also Part 5) recognises a difference between sections that it terms ‘compact’, where the plastic moment resistance of the section can be developed, and those that it terms ‘non-compact’, where local buckling of the web or flange limits the moment resistance to that at first yield in an extreme fibre.

7.1.1 Requirements at ultimate and serviceability limit states

Part 3 states that beams should be designed to satisfy the requirements at ULS and that, in determining bending resistance at ULS, the effects of shear lag may be ignored.

Clause 3/9.2.3 states that the requirements at SLS need only be checked for the structural steel elements when one (or more) of the following conditions arise:

(i) Shear lag is significant; this arises when the effective breadth ratio is less than a restricting value given by Clause 3/9.2.3.1(a).

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(ii) Tension flange stresses have been redistributed at ULS in accordance with Clause 3/9.5.5. This redistribution is not usually employed in railway bridges.

(iii) An unsymmetric beam is designed as a compact section at ULS (this may be the case for composite deck-type bridges and for cross girders in half through bridges).

For composite beams, Part 5 requires that the stresses in reinforcement and concrete should be checked at SLS, taking account of any co-existent stresses due to local bending of the slab (Clauses 5*/5.2.4 and 5*/5.2.6). Calculation of stresses at SLS should allow for the effects of any shear lag (Clauses 3/8.2 and 5*/5.2.3). Crack widths in tensile regions of the slab should be checked at SLS.

7.1.2 Section classification

Definition of compact and non-compact sections

Compact sections are those where 'the full plastic moment can be developed before, and maintained after, the onset of buckling' (Clause 3/9.3.7.1). Limits are given in Part 3 for breadth/thickness ratios for which this requirement is deemed to be true (Clause 3/9.3.7).

Part 3 does not explicitly define "non-compact" sections, it merely refers to them as the alternative to compact sections. There is, nevertheless, a limit to the outstands of compression flanges that applies to non-compact sections (Clause 3/9.3.2.1); this limit allows a flange to develop its full yield strength, although it cannot sustain plastic strain like the flange of a compact section.

Design implications for compact sections

The following remarks apply chiefly to deck-type bridges, although they would also apply where cross girders in a half through bridge were designed to act with a slab as a compact section (although this is not usual). In half through construction, there is usually little or no advantage in utilising compact capacity for the main girders.

When a plastic moment is developed in a beam, the stress distribution through the section takes the form of rectangular blocks rather than the usual 'triangular' distribution (Figure 7.1). The amount by which the plastic moment exceeds the elastic moment when yield is just reached depends on the relative proportions of web and flange; the ratio between the two moments is commonly known as the shape factor.

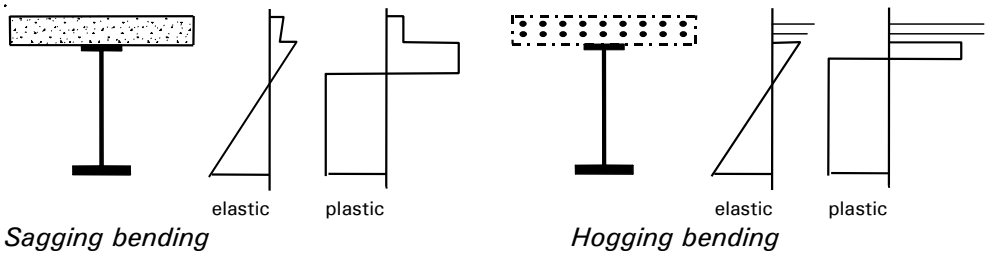

Figure 7.1 *Stress distributions in composite beams*

Because the bending resistance of compact sections at ULS is determined by redistribution of the stresses at any cross section, the design requirement for a compact section is simply that the plastic moment capacity is adequate to carry the total moment acting at that cross section at ULS (Clause 5*/6.2.2). It is not

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necessary to check the summation of the separate ULS stress distributions for each stage of construction (see Section 7.1.4).

Although the strength of compact sections is determined from a plastic stress distribution, the distribution of bending moments should still be determined by elastic analysis. Further, because in continuous bridges the design loading for the two regions (hogging and sagging) differs, each region may be designed on its own merits; compact moment resistance may be used in midspan regions of deck type bridges when non-compact resistance is used at intermediate supports.

If the beam is compact throughout the span, and the member is not too slender, the effects of differential temperature, creep and settlement may be neglected at ULS [see Clause 3/9.2.1.3(iv)]. This allowance appears to be based on the presumption that the secondary moments (due to temperature gradient and shrinkage) can redistribute in such cases.

Unsymmetric beams designed to plastic moment capacity will reach yield in one flange before the other and before the full moment resistance is developed. Consequently, the stress in the more highly stressed flange must be checked elastically at SLS (Clause 3/9.9.8) to ensure that yield is not exceeded at that state and taking into account construction in stages for composite beams (see Section 7.1.4). The effects of temperature gradient and shrinkage must then be included.

To achieve plastic moment in a composite section, the shear connectors must be capable of transferring an appropriate shear between the beam and the slab.

It should be noted that steel beams that are compact when acting compositely with the slab might not be compact when acting alone during construction. In such a case, the checks for the construction stage must be made on the basis of non-compact sections.

Design implications for non-compact sections

The design strength of non-compact beam sections is determined by essentially elastic stress distributions and limiting stresses. In most cases, therefore, checks need only be made at ULS. Checks at SLS are required in only a few circumstances (Clause 3/9.2.3.1).

A non-compact composite section must provide sufficient moment resistance for the total moment acting at that section (determined as a summation, as explained in section 7.1.4). However, because a non-compact section is not able to redistribute stresses, a check must also be made on the summation of the elastically determined stresses.

7.1.3 Resistance of beams to buckling

Lateral torsional buckling (LTB) is a well-known phenomenon in slender beams that are free to buckle between positions of restraint. True lateral torsional buckling only occurs where the beam is entirely free between discrete restraint positions; where there is a continuous deck at tension flange level, or some intermediate level between flanges, the mode is actually *lateral distortional buckling*, but the code treats this as lateral torsional buckling, for simplicity.

In design, lateral torsional buckling behaviour is taken into account by use of a ‘buckling curve’ such as that illustrated diagrammatically in Figure 7.2.

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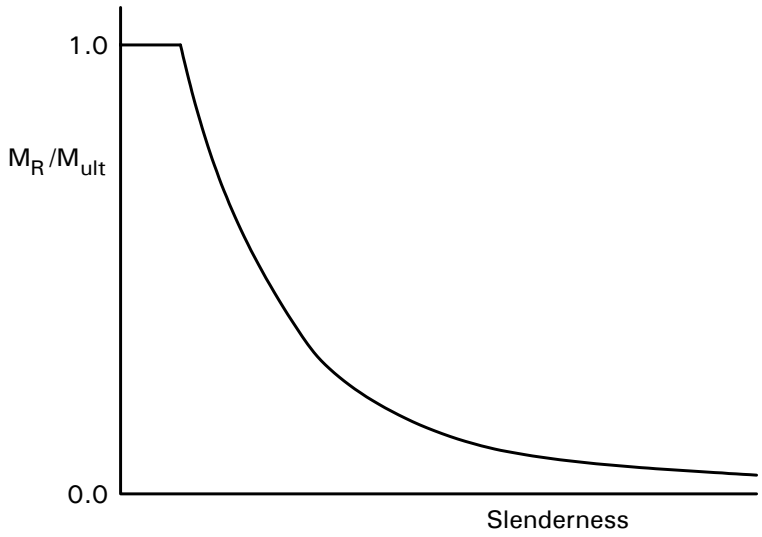

Figure 7.2 *Typical LTB buckling curve*

The proportion of the bending resistance of the cross section that can be developed depends on the beam's LTB slenderness, which can be expressed in terms of an effective length and various section properties. Part 3 gives rules for determining the effective length of a beam in bending, depending on the restraint to the compression flange. It provides for four different ways in which a compression flange can be stabilised against buckling laterally out of its plane.

Continuous direct restraint

The first and most direct form of restraint is when the flange is connected directly to a deck (e.g. top flange of a single span composite beam and slab bridge and the midspan portion of a continuous composite bridge). Then the effective length is zero (Clause 3/9.6.4.2.1) and M_R simply equals M_{ult}. (See Section 7.2.1 for definitions.)

Discrete intermediate lateral and torsional restraints

The second form of restraint is mainly only seen during construction of deck-type bridges, when the compression flanges of the bare steel **I** beams are typically restrained at discrete positions by means of lateral or torsional restraints.

Where there is plan bracing to the top flange of a simply supported span, it provides discrete lateral restraint that are so stiff that they are fully effective, and then the buckling effective length is equal to the spacing of the restraints (Clause 3/9.6.4.1.1.1). Similarly, a direct connection of the flange to another structure (say a parallel length of completed deck) would be fully effective.

However, plan bracing to the compression flange of deck-type spans during construction is not essential if stiff torsional restraints are provided between two beams at discrete intervals. This effectively creates a 'torsional spring restraint' that modifies the buckling mode between support positions. The effective length with this form of restraint is given by Clause 3/9.6.4.1.2. If the torsional stiffness provided by the action of the pair of beams is sufficiently high, the effective length can be restricted to about one-third of the span, but any further restriction cannot practically be achieved (see the 'distributed restraints' curve in Figure 8 of Part 3).

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The bottom flanges adjacent to intermediate supports of deck-type spans are often restrained at discrete positions by cross bracing acting in conjunction with the deck slab. This form of restraint is usually so stiff that it provides fully effective discrete lateral restraints.

Discrete U-frame restraints

In half through construction (other than the 'Standard Box Girder'), the compression flanges are restrained at discrete positions not by cross bracing but by the stiff frames formed by the cross girders (acting compositely with the deck slab) and the web stiffeners. These are referred to as discrete U-frames. The stiffnesses of such restraints are evaluated according to Clause 3/9.6.4.1.3 and the effective length is determined according to Clause 3/9.6.4.1.1.2.

In determining the stiffness of a U-frame, the factor f for the flexibility of the connection is usually the dominant parameter. Unfortunately, the 'example' connections shown in BS 5400-3 for the three classes of flexibility do not bear much relation to typical U-frame connections. Guidance on the appropriate f factor for endplate connections is given in Section 10.

Continuous U-frame restraint

In some small bridges, the compression flange may receive restraint from the U-frame action of a slab (at the level of the tension flange) with an unstiffened web plate. This is termed continuous U-frame restraint and the effective length is determined (Clause 3/9.6.4.2.2) by considering the restraint as a series of U-frames, each of unit length.

Beams without intermediate restraint in half through bridges

Because the use of U-frame action in half through bridges leads to additional design requirements and has fatigue and other drawbacks, for shorter spans and where space permits, it may sometimes be preferable to widen and thicken the main girder compression flanges and to rely only on torsional restraint at the ends of the girders. No intermediate U-frames would be created. If this choice is made, the consequences on installation arrangements (notably the stability of girders during erection) must be considered.

Additionally, in the absence of intermediate restraint, alternative measures should be provided to ensure that the main girder has reasonable robustness in the event of accidental impact from derailed trains. It may be necessary to consider protective elements or to enhance the top flange to resist lateral impact loading.

7.1.4 Composite beams – construction in stages

Composite deck-type bridges are usually built in stages: the steel girders are erected first; the concrete slab is the cast over one part of the bridge; the slab is then cast successively over the other parts until complete; finally, the waterproofing, ballast, services and parapets are added. At each stage, the structure is different, and the distribution of bending moments (in a continuous bridge) and the interaction between steel and concrete elements are different. Additionally, the response of the completed structure to live load is different again, because the concrete is stiffer in response to short-term loading.

Consequently, for deck type bridges, it is necessary to determine moments and shears at the various different cross sections as summations of a number of values. This summation is necessary even for beams that are deemed to be

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compact, although in those cases some of the load effects can be neglected, because it is assumed that they can redistribute within the structure at ULS.

The total stresses and strains in the fibres of a composite beam where the main girders are unpropped and the deck slab is added in stages are determined as the summations of the distributions for each stage (and similarly for short- and long-term loads). This summation is shown diagrammatically in Figure 7.3. The position of zero stress will therefore not necessarily correspond with any particular neutral axis level.

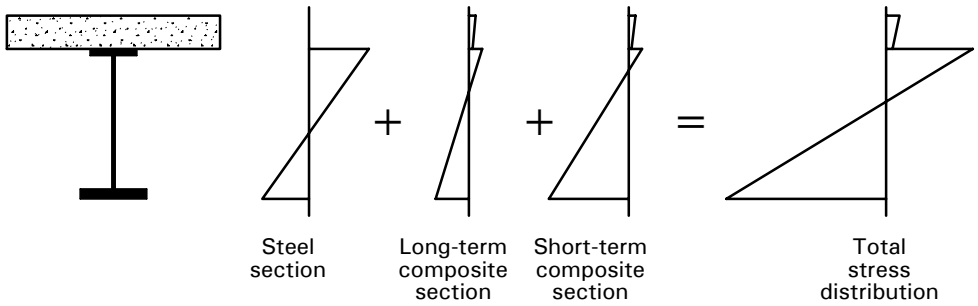

Figure 7.3 *Summation of stresses for unpropped staged construction*

Where SLS must be checked as well as ULS, the stress distributions for SLS and ULS must be calculated separately, each using its appropriate set of partial factors for the various loads.

As mentioned in Section6.7, in half through bridges it is usual to neglect the beneficial action of the deck slab in tension at ULS but it is necessary to consider the effect of composite action on the slab and its connection to the cross girders at SLS and for evaluation of fatigue effects . This will involve some consideration of staged construction and summation of stresses, in a similar way to that for deck type bridges although, where the slab is cracked in tension, there is no distinction between long- and short-term effects.

7.1.5 Restraint of beams at supports

Bracing or some other restraint system is required to restrain the primary members (the longitudinal beams) and to provide a load path for transferring horizontal forces (transverse to the main beams), particularly at support positions.

Generally, beams are required (Clause 3/9.12.5.1) to be provided with torsional restraints at supports, because they will have been designed to resist lateral torsional buckling. These torsional restraints must also be capable of transferring lateral loads such as wind load, nosing and centrifugal loads down to the level of the bearings.

In half through bridges, torsional restraint at the support, although usual, is not always essential, if other means of restraint is provided within the span. See further discussion in Section 7.4.1. Lateral forces are usually transferred directly from the bottom flange to the bearings.

7.1.6 Thickness of weathering steel

Where unpainted weathering steel is used, an additional thickness of steel (not included in the calculation of section resistance) should be provided to cater for the loss due to slow rusting (the so-called rust ‘patina’). The additional

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thickness on each exposed surface depends on the atmospheric environment; recommendations are given in clause 4.5.6 of BS 5400-3.

7.2 Design resistance of beams without longitudinal stiffeners

Bending and shear resistances of a beam without longitudinal stiffeners are assessed separately and the effects of interaction between the two are taken into account by expressions that define a limiting envelope for the combined load effects. Naturally, the combination of bending and shear is greatest at internal supports of continuous spans, though it is still necessary to check other regions for combined effects, particularly where there is a change of section.

7.2.1 Bending resistance

In BS 5400-3, the bending resistance M_D of a beam without longitudinal stiffeners is expressed in terms of the limiting moment of resistance M_R divided by partial factors γ_m and γ_{f3} (Clause 3/9.9.1.2). The value of M_R is determined by the strength of the cross section M_{ult} multiplied by a reduction factor (where appropriate) to allow for lateral torsional buckling (Clause 3/9.8 and Figure 11).

For non-compact sections, the value of M_{ult} is the strength at first yield in an extreme fibre of the cross section (determined from the effective section properties, with no allowance for shear lag but reduced for holes and slender webs, Clause 3/9.4.2). For compact sections, the value of M_{ult} is the plastic moment capacity of the effective section, M_{pe}.

The use of this approach for compact sections, using a plastic section property and a curve that is essentially dependent on elastic buckling, is difficult to envisage in physical terms, but calibration against experimental tests confirms that it is a reasonable evaluation.

For composite sections in deck-type bridges, the elastic bending resistance is not normally governed by the strength of either the concrete or the reinforcement, but the plastic moment capacity does depend on the strength of those elements. (And, as noted in Section 7.2.2, the value of M_{pe} is needed even for non-compact sections.) See Section 7.5 for comment on values of design strength for concrete and reinforcement.

7.2.2 Determining the limiting moment of resistance

Figure 11 gives the value of the ratio M_R/M_{ult} for the value of a 'modified slenderness parameter'. The evaluation of the modified slenderness parameter requires the designer to calculate first the effective length for lateral torsional buckling (ℓ_e), second the slenderness parameter λ_{LT} and third a modification to take account of the yield strength and the relationship of M_{ult} to M_{pe}. The parameters involved in this process depend on bracing arrangements and moment variation along the beam (Clauses 3/9.6 and 3/9.7).

Effective length

The effective length for lateral torsional buckling is given by Clause 3/9.6, depending on the restraint to the compression flange, as discussed in Section 7.1.3.

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Slenderness λ_{LT}

Once the effective length has been determined, the slenderness is calculated according to Clause 3/9.7. The expression for λ_{LT} includes a parameter η that allows for any moment variation over the half-wavelength of buckling. Where there is little variation, the parameter is close to 1.0; at intermediate supports of continuous deck type bridges, the variation over the half wavelength adjacent to the support can make a significant reduction in the value of λ_{LT} (although η can always be taken conservatively as 1.0).

Modified slenderness

As mentioned above, the determination of M_R involves the use of the modified slenderness parameter and a buckling curve. The modification to the value of the slenderness parameter λ_{LT} is the multiplication by two ratios. The first ratio is the value $\sqrt{\sigma_y / 355}$. This allows for the actual strength of the material (Figure 11 of Part 3 is drawn for the specific value of 355 N/mm^2 for the yield strength).

The second ratio is the value $\sqrt{M_{ult} / M_{pe}}$. For compact sections, this value is unity. For non-compact sections, the value is less than unity: although it may seem surprising that this requires the calculation of M_{pe} for a non-compact section, the modification is needed because the buckling curve in Figure 11 has been calibrated, for both compact and non-compact sections, against the value of M_{pe}.

The 'buckling curve' in Part 3

Figure 11 in Part 3 is in two parts, one for welded members and one for non-welded or stress-relieved members (the allowance for residual stress is different between the two parts). Also, in each Figure, three curves are given, for different values of the ratio effective length/half-wavelength of buckling (ℓ_e/ℓ_w); the need to consider this ratio arises from the theoretical basis for the 'buckling curve'.

The derivation of a buckling curve assumes an initial imperfection and this is usually expressed as a function of the effective buckling length (the Perry-Robertson approach). But a strut or a beam can buckle in only a finite number of half waves and the half-wavelength of buckling may be somewhat greater than the effective length. This is taken into account in Part 3 by the use of a series of buckling curves, for different ratios of half-wavelength to effective length. Where the load effect varies over the span length, it is also important to take account of the actual half waves, so that the most severely loaded half wave is checked.

For an 'ordinary' case, with fully effective restraints, the ratio ℓ_e/ℓ_w is 1.0.

For a more detailed explanation of the determination of slenderness and limiting moment of resistance, see *Commentary on BS 5400-3:2000, Code of practice for the design of steel bridges*[46].

7.2.3 Treatment of staged construction

As mentioned in Section 7.1.4, the stresses through a non-compact section that is constructed in stages have to be determined by summation of several stress distributions. (This applies also to compact beams treated as non-compact at

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SLS.) The resulting stress variation is therefore non-linear and discontinuous, and could not be derived from any single application of loads to a single section.

For these cases, verification of adequate bending resistance at ULS depends on two requirements: first, that the total moment (the sum of all the stage moments) should not exceed M_D (see Section 7.2.1 for comment on bending resistance); and second, that the total accumulated stress at any fibre should not exceed the factored yield strength (Clause 3/9.9.5.4). This second requirement also applies to the SLS check on compact sections.

7.2.4 Shear resistance

Economic design of longer spans, usually minimising the cross-sectional area of the steel girder, results in relatively thin webs. Unfortunately, an unstiffened slender web is unable to develop full shear yield resistance, because its capacity is limited by buckling. However, when the web is provided with transverse stiffeners, the buckling resistance is increased. The increase arises firstly from the constraint of the rectangular panel and secondly because some of the shear is carried by tension field action. The mechanism for the latter is discussed in Reference 46.

The design shear resistance V_D (Clause 3/9.9.2.2) is the shear capacity of the web panel, including the contribution to the tension field action from the flanges.

Intermediate web stiffeners are an effective means of increasing the limiting shear stress when the web depth/thickness is in excess of about 75. Typically, intermediate web stiffeners are provided at a spacing of between 1.0d and 2.0d.

The same rules for calculating shear resistance apply to webs in both compact and non-compact sections. However, the webs in compact sections are less slender, by definition.

Form of intermediate transverse web stiffeners

The most common form of transverse web stiffening is a flat plate welded on one face of the web. Such stiffeners are typically used in deck type construction when the web depth/thickness (d/t) exceeds about 50. A web d/t up to as much as 200 may be used in a web stiffened only by transverse stiffeners. These stiffeners are usually attached to the top flange but not to the bottom flange.

In half through construction, U-frame action normally depends in part on the stiffness and strength of the web stiffeners that align with the cross girders. They must therefore be designed for their combined actions as web stiffeners and U-frame members and must be spaced to suit the cross girders. Generally, cross girders are likely to be more closely spaced than is necessary either for stiffening of the slender web or for U-frame action; it is common to position stiffeners at every second or third cross girder, with the other cross girders simply bolted to the plain web. In deep girders, Tees rather than flats have been used on older bridges, but this is not common now.

7.2.5 Bending–shear interaction

Under combined bending and shear, a bending resistance M_f equal to that provided by the flanges alone, ignoring any contribution from the web, can be

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sustained at the same time as the shear resistance V_R if contribution from the flanges is ignored. Further, it has also been shown that the full bending resistance M_D can be developed if the shear is not more than half of V_R, and that full shear resistance V_D can be developed if the bending is not more than half of M_f. These limits to the interaction are expressed in Clause 3/9.9.3.1 and are shown graphically in Figure 7.4.

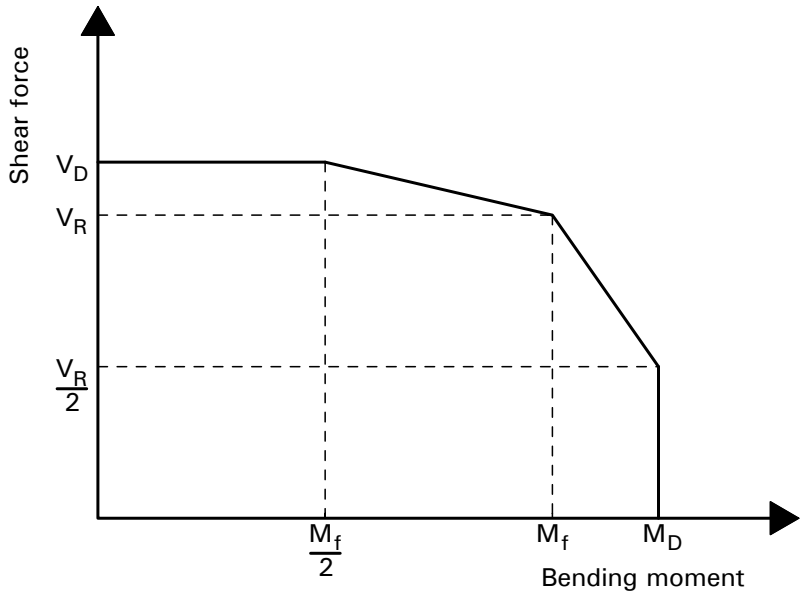

Figure 7.4 *Interaction between shear and bending resistance*

Bending-shear interaction is checked for the worst moment and worst shear anywhere within a panel length (i.e. between intermediate web stiffeners), rather than at a single section. This is only slightly conservative for panels adjacent to internal supports of continuous beams but seems rather onerous for midspan regions of beams of compact section in deck-type bridges.

For beams constructed in stages, the moment acting on the section should be taken as the total moment for sections designed as compact, but for sections designed as non-compact an effective bending moment must be derived for use in the interaction formulae. This is obtained by multiplying the extreme fibre total stress by the modulus for that fibre in the section that is appropriate to the stage of construction being checked (see Clause 3/9.9.5.5). The designer should take care to ensure that the fibre for which total design stress is used to determine the equivalent bending moment is the same fibre that determines the bending resistance.

7.2.6 Longitudinal shear connection

In the following discussion, references are to clauses in the Yellow Document, rather than in BS 5400-5 (see Section 5.5.2).

In composite sections, to provide the necessary shear transfer between the steel girder and the concrete slab, shear connectors are required. The shear flow varies along the length of the beam, being highest near the supports, and it is customary to vary the number and spacing of connectors commensurately, for economy. The most commonly used form of connector is the headed stud, though channel connectors are sometimes used.

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Shear connectors must be designed to provide static strength and for fatigue loading. With non-compact sections, the required resistance at SLS generally governs the design for static strength. Shear flows should be calculated at supports, at midspan and at least one position in between, i.e. quarter points. The ULS need only be considered for non-compact sections when there is uplift applied to the connector or redistribution of tension flange stresses (Clause 6.3.4). Fatigue may well govern the spacing of connectors in midspan regions.

For compact sections, the shear resistance is checked at ULS by calculating the shear flow on the basis of elastic section properties, and assuming the concrete to be uncracked and unreinforced (Clauses 6.3.1 and 5.3.1). This requirement ensures in practice that there is sufficient connection to develop the plastic moment resistance.

The nominal static strength of shear connectors is given in Table 7 and the design static strength in Clauses 5.3.3.6 and 6.3.4. The design of the connectors must provide a resistance per unit length of at least the maximum design load shear flow over 10% of the length of the span each side of a support. In other parts of the span, a series of groups of connectors at constant spacing may be used to provide a 'stepped' resistance, subject to the provision of sufficient total resistance over each length. The maximum calculated shear flow within the length of any such group must not be more than 10% in excess of its design resistance per unit length. The design static strengths are given in Clauses 5.3.2.5 and 6.3.4, and the design procedures in Clauses 5.3.3.5 and 6.3.4. Note that document GC/RC5510 Appendix B modifies the partial factors on strength for shear connections and allows connectors to be grouped with wider spacings between groups.

It should be noted that although the code requires consideration of uplift when there is tension field action (Yellow Document Clause 5.3.3.6), there are no provisions for the determination of its value, its point of application or the length over which it may be resisted. Such uplift is usually ignored by designers.

Transverse reinforcement is required in the slab to provide shear resistance at ULS in a similar manner to the requirements for shear stud spacing (Clause 6.3.3). The required area of reinforcement is usually provided in multiple beam construction by continuity from midspan of the slab of the bottom layer of transverse reinforcement.

In half through construction, as mentioned in Section 6.5.2, it is usual to neglect composite action at ULS but the adequacy of the longitudinal shear connection at SLS should be checked. The shear connection may be provided only by the cross girder connections, in which case they must be designed for the combined effects, or shear connectors may be provided along the web to help transfer the longitudinal shear. The fatigue effects of longitudinal shear forces should also be checked. For cross girders that project beyond the edge of the deck (for example as shown in Figure 10.17), the design should take into account bending effects generated in the cross girders by longitudinal shear effects.

7.3 Filler beams

The design of filler beam construction should generally be in accordance with Clause 8 of the Yellow Document. However, the document does not give any specific recommendations for determination of transverse moments for railway

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bridges (clause 5*/8.3 applies only to highway bridges). In the absence of such recommendations, a grillage model may be used; the application of railway loading along the lines of the rails is a more direct way of determining the transverse load effects than the means of allowing for the transverse distribution of highway loading that is used in clause 5*/8.3.

The bond strength between the steel beams and the concrete is also only given in Part 5 for highway and footway bridges. For railway bridges, it would seem appropriate, for 'regular' filler beam construction to use the same value. By 'regular' is meant the use of longitudinally spanning filler beams, with no tensile effects transverse to the beams. However, filler beam construction is used as the deck slab of half through bridges and in such circumstances there is tensile stress that is transverse to the cross girders. It is considered unreliable to assume any bond stress in such circumstances (in early Z type bridge decks significant cracks were found to occur parallel to and at every cross girder).

For 'regular' filler beam bridge design, the rules in Part 5* can generally be used for railway bridges. It is advisable to discuss the design basis for a filler beam deck with the railway authority at an early stage; reference to more recent documents than BS 5400 may be agreed (for example, guidance can be obtained from a UIC publication[55] and detailed rules for filler beam decks are given in the draft Eurocode 4[30]). Use of rules other than BS 5400 will require the acceptance as a 'Departure from Standard' by the railway authority.

7.4 Restraint systems

7.4.1 Bracing in half through bridges

Design forces for U-frames

Intermediate U-frames are to be designed to resist horizontal forces F_R acting normal to the compression flange at its centroid (Clause 3/9.12.3.2). Equal and opposite forces are assumed to act on the two flanges. In addition, horizontal forces F_C are to be applied, in the same manner as F_R, to U-frames that are subject to vertical loading.

To determine the magnitude of F_C, it is necessary to know the rotation at the ends of the cross girders of the U-frame, assuming them to be simply supported. Further, the value of the rotation is not the 'absolute' value but the value relative to the U-frames either side. The relative value is needed to represent the fact that if all cross girders rotated the same amount, there would simply be a bodily horizontal displacement of the flanges and F_C would be zero. The relative rotation is then used to determine the value of F_C, depending on the stiffnesses of the U-frame and of the flange in horizontal bending (Clause 3/12.3.3).

A similar requirement applies to end U-frames, except that the forces are defined in clause 3/9.12.5.2 as F_S and F_L.

Note that the force F_R may be in either direction (inward or outward) and that whilst the greatest value of F_C is likely to be outward (restraint to the U-frame under greatest load), adjacent frames would experience inward forces and thus it is easier to assume that design value of F_C may be in either direction.

Additional loads on U-frames will occur where walkways are attached to the main girder web stiffeners that form part of the U-frame. U-frames must also

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be checked for horizontal loading arising from the action of the main girders as robust kerbs.

Where Z-section girders are used, the asymmetry of the main girders leads to additional effects (twist and lateral displacements). These effects are usually resisted by the U-frames (in addition to the resistance they provide against the effects of the F_R and F_c forces).

Non-uniform beams and restraint systems

Code rules are generally expressed for relatively simple uniform situations, because the formulation of rules to express complex behaviour can only cover standardised situations. In half through construction there are two aspects of non-uniformity that often give rise to questions.

The first is when the beam cross section changes significantly along the span. This is quite likely when doubler plates are used to enhance the capacity in midspan and are then curtailed somewhere about a quarter or a fifth of the span from the support. If the effective U-frames remain the same throughout the span and there are three or more half waves, then an analysis on the basis of midspan properties will still give a reasonable value for effective length of midspan regions. But this value will be an overestimate of the effective length for the end regions (the main beam being weaker relative to the U-frames, a greater effective length would be given if its properties were used to determine effective length). Use of the effective length derived for midspan in conjunction with beam properties for the end region should be conservative for the end part of the span. (It is not the case that the weaker compression flange in the end region offers less lateral restraint to the midspan region, because, in the model, there is zero lateral moment and zero displacement at the nodes of the half-wave deflected profile.)

The second is the restraint provided at the ends of skew bridges. The endmost intermediate frames are likely to be L-frames rather than U-frames and, for high skews, there may be no end restraint at all. End restraint is not always needed for obtaining an adequate value for the effective length (this is a change from the rule in the previous issue of Part 3), but the question arises of how to meet the design rules for restraints. It is suggested that restraint forces F_S are determined in accordance with 3/9.12.5.2 for the end of the beam and that, if there is inadequate restraint (i.e. the bearing cannot provide the restraint and there is no restraint from a trimmer), the beam is then checked for its adequacy to carry the force (in plan bending of the flange) back to the nearest L-frame. The L-frame is then checked for its adequacy to carry the force. For the determination of effective length, L-frames will be stiffer than the equivalent U-frame, so the assumption of uniform stiffness of intermediate U-frames should be conservative for that purpose.

Additionally, the determination of appropriate F_c forces and other load effects generated by the flexing of the whole structure is difficult for skew bridges, particularly near the supports, because the vertical deflections and rotations are different at the two ends of the cross girder. Account needs to be taken of differential deflection and of restraint provided by the torsional and warping stiffness of the main girder.

In determining the forces on L- and U-frames towards the end of main girders, it is necessary to consider the level of torsional restraint provided by the main girder to the cross girder that results in partial fixity to the cross girder. The

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resultant load effects are increased in skew bridges where one end of the L- or U-frame is near a fixed support and the other end is subjected to main girder vertical deflection. The connection, intermediate stiffener and end of the cross girder are designed to resist these effects. It is normal practice to neglect the beneficial effects of fixity to the end of the cross girder when designing the central portion of the cross girder".

7.4.2 Bracing in deck type bridges

For simply supported spans, intermediate bracing is not needed to restrain the main beams, except possibly for the construction condition. For continuous spans, beams are often braced adjacent to intermediate supports to provide restraint to the compression flange. To assist in resisting accidental soffit impact loads, it may be necessary to brace pairs of girders, typically at 3 to 4 m centres.

Bracing across the full width of a multiple girder bridge will tend to act as a stiff transverse element when loads are other than uniform across the deck width. The members of such bracing will be subject to load reversal when different parts of the deck width are loaded (i.e. loading from one track, then from the other); the bracing connections may be subject to significant fatigue loading. A different arrangement with multiple beam bridges is to brace the beams in pairs, with no bracing in the bays between the pairs. This makes the transverse bracing 'non-participating' in transverse load distribution.

Stability of the compression flange must be achieved during construction as well as in service. In sagging regions, the resistance of a composite beam at ULS is usually governed by the tensile yield of the bottom flange. During concreting, with the weight of wet concrete carried only by the steel beams, lateral torsional buckling and the stability of the top (compression) flange may well govern the design. Adequate bracing to the top flange must be provided for this condition, although it may be temporary and can be removed after concreting. Transverse bracing (between a pair of beams) is often sufficient for this purpose, although plan bracing to the flanges may be required for carrying lateral wind loads in longer spans (but it adds to the complexity of fabrication and erection, and should be avoided where possible).

7.5 Material strength and notch toughness

7.5.1 Steelwork

Strength

The most cost-effective grade of structural steel (in terms of strength per unit cost of material) is grade S355 to BS EN 10025[2]. That grade is almost exclusively used in highway bridges and for a large number of railway bridges. However, where the design of a railway bridge is governed by very onerous fatigue conditions or serviceability limits (deformation, dynamic response or control of cracking), grade S275 may be sufficient.

Notch toughness

All parts of structural steelwork are required to have adequate notch toughness, to avoid the possibility of brittle fracture (Clause 3/6.5). Brittle fracture can initiate from a stress concentration when loading is applied suddenly, if the material is not sufficiently 'tough'. The degree of toughness required is expressed as a Charpy impact value (determined from a test carried out on a

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sample of material) and the requirement depends on the thickness of the material, its minimum temperature in service, the stress level and rate of loading.

The design requirements are expressed in Part 3 in terms of a limiting thickness for the given conditions and specified material. If the thickness of the part does not exceed the limit, there is sufficient toughness.

The most obvious condition that needs to be considered is the lowest temperature that the steel will experience. Minimum bridge temperatures are specified in Part 2 (or BD 37), Clause 5.4.3. For steel railway bridges (i.e. 'Group 2' in clause 5.4.3) minimum bridge temperatures are typically –19° C in England and –23° C in Scotland.

Other conditions relate to the stresses locally at the detail. Special '*k*-factors' to account for stress level (k_σ), rate of loading (k_s), stress concentration (k_g) and potential fracture initiation site (k_d), are given in Clause 3/6.5.3. Multiplying all four factors together gives a single overall factor *k*.

Steel material, to one of the Standards listed in Clause 3/6.1.2, will have a specified minimum Charpy impact value at a given test temperature. Commonly, steel grade S355J2 to BS EN 10025[2] is chosen for bridge steelwork, and this has an impact energy of 27 J at –20° C. (The impact energy is indicated by the 'J2' part of the designation.)

The 'maximum permitted thickness' of the steel part is given in Clause 3/6.5.4 in terms of an equation relating the overall factor *k* and design minimum temperature, the chosen material yield strength and the Charpy test temperature for the chosen material grade. The maximum thickness is directly proportional to the value of *k*.

For the specific case of $k = 1.0$, a simple tabular presentation of limiting thickness for the grades of material covered by Clause 3/6.1.2 is given in Table 3c of Part 3. For grade S355 steel, the table gives thickness limits at –20°C of 50 mm for grade J2, 60 mm for grade K2 and 86 mm for grade NL (to BS EN 10113-2). In the absence of gross stress concentrations and sudden loading, the actual limit for a component depends on the values of factors k_σ and k_d.

Factor k_σ varies from unity for parts under significant tensile stress to a value of 2 for parts always in compression (see Table 3b of Part 3). Clearly, this means that parts in compression can be thicker than parts in tension, for a given toughness grade.

The value of factor k_d depends on the likely flaw size at the potential fracture initiation site (i.e. where a brittle fracture would start). Although brittle fracture is a different phenomenon from fatigue, details susceptible to fatigue are also more susceptible to brittle fracture. For that reason, Table 3a of Part 3 refers to the BS 5400-10 fatigue class details in determining the k_d factor. Some class F details and all class F2 and G details lead to a k_d value less than unity

The combination of the two factors can lead to a *k* factor as low as 0.5 for tension flanges but up to 2 for compression flanges. Consequently, the thickness limits in Table 3c may be halved or doubled for a particular detail; it

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may be difficult to obtain a suitable grade of thick S355 material for the former case.

When the required flange thickness would exceed the toughness limit, doubler plates can be used, each plate being about half the thickness of a single plate. This option is also sometimes chosen to avoid the use of the higher quality NL grade in thick plate. Additionally, Network Rail's current practice is to limit plate thickness to 75 mm, even if thicker flanges could be chosen of a grade to give sufficient toughness. (It is understood that thicker material may be acceptable to Network Rail in specific cases). Note however that the termination of doubler plates is a class G detail and the toughness requirement is likely to govern the positions where doubler plates can be curtailed in tension flanges.

It is not necessary to use the same grade throughout the whole structure – thick tension flanges could be K2 grade while the remainder is J2, for example – but the designer should be aware of the possibilities for confusion if more than one grade is used.

Note also that, for $k = 1$, Table 3c is slightly conservative for higher values of the thickness limit. See GN 3.08[47] for a modified version of Table 3c for use with thicker material.

7.5.2 Reinforced concrete

Concrete grade 40 is most commonly used for bridge decks and reinforcement is grade 460 to BS 4449.

7.6 Slab design

Slabs should be checked for adequacy at both ULS and SLS for bending and shear, including the combination of local and global effects where the slab is designed to participate compositely with main members. Derailment effects must also be checked at ULS.

Limiting stresses and crack widths are given in the Yellow Document.

In half through bridges, local (longitudinal) bending arises from spanning transversely between cross girders and from relative deflection of adjacent cross girders. The slab will also be subject to longitudinal tensile strains and these must be considered when checking the crack widths at SLS and the resistance to local bending at ULS. Tension stiffening may be taken into account , as it has a beneficial effect on the bending resistance of slabs under tensile loading. See comment on global analysis in Section 6.5.2.

Slabs spanning transversely in half through decks will only act compositely with cross girders if there is adequate shear connection. Bond alone cannot be relied upon to generate composite action; either sufficient shear connectors must be provided or sufficient reinforcement passed through holes in the webs of the cross girders. Where there is no composite action, the transverse bending strength at ULS may be taken as the sum of the separate resistances of the slab and cross girder; at SLS loads are shared on the basis of equal deflection of the slab and the cross girder.

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7.7 Interfaces with substructures

The design of new substructures or the checking of existing substructures must be compatible with the design assumptions used for the superstructure. (Particular attention may be needed to ensure compatibility when different design organisations are involved.) Where an existing substructure is to be reused, it is important to ensure (by adequate survey and study of existing documentation) that the substructure is suitable for supporting bearing configurations, positions and loadings that may be significantly different from those which existed previously. Particular aspects to check are, for example, hidden vaults or thinner abutment walls that may exist beneath new bearing positions. Generally, the objective is usually to ensure that the maximum load effects from the new structure are no more than from the existing structure.

Often, new sill beams are used to replace non-existent or localised bedstones (or even simply to speed the installation process. New sill beams may be used to span over or beyond weaknesses in existing substructures. Allowance should be made in their design for tolerance in the relative position of new bearings and existing substructure, to ensure that proper fit is achieved at site. The design of sill beams must be coordinated with that of any ballast walls, other ballast retention features, arrangements for bearing inspection/replacement and walkway approach details. A key consideration is ensuring satisfactory waterproofing and drainage details that will keep water away from the bearings.

Document GC/RC5510, clause 7.5.7 and Appendix C, gives detailed advice on the re-use of existing substructures.

7.8 Bearings

7.8.1 Bearing types

General guidance on the types of bridge bearings that can be used with steel bridges is given in RT/CE/C/008 Section 100GN[12] and GN 3.03[47]. Traditionally, line rocker bearings and roller bearings have been used with longer (i.e. span over 15 m) half through plate girders. Elastomeric pot bearings are now used in many instances, particularly with composite beam and slab bridges.

Fabricated line rocker bearings

Line rocker bearings allow rotation to occur about an axis along the line of contact whilst providing rotational restraint about the horizontal axis square to the line of contact. They have the advantage of being highly durable, not being reliant on the use of moving parts or on low friction or compressible materials with limited life. The facility to restrain rotation about one axis can be advantageous in achieving overturning resistance or stability against buckling at supports.

Line rocker bearings are usually aligned square to the girders although for small skews, alignment with the skew may be more suitable (see further comment in GN 3.03[47]). They are less suitable on highly skewed bridges, except for the standard box girder bridges (where they provide the only torsional restraint to the boxes and are aligned square to the girders).

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Roller bearings

Roller bearings may be needed where torsional restraint to the beam is required (in similar circumstances to those where line rockers are needed), but proprietary roller bearings are expensive. Purpose-mode roller bearings can be designed and fabricated but specialist input should be obtained.

Pot bearings

Elastomeric pot bearings are now commonly used for railway bridges. They provide rotational freedom transversely and longitudinally and are much cheaper than an alternative spherical bearing. They are particularly useful in highly skew half through bridges.

7.8.2 Articulation

The articulation of a bridge – the arrangement of restraints provided at the various bearings – is a subject that requires careful consideration. Horizontal forces need to be properly restrained but expansion and contraction need to be allowed for. In railway bridges, the longitudinal track bridge interaction needs special consideration, see clause 12.1 of GC/RC5510. General guidance on bridge articulation is given in GN 1.04[47].

7.8.3 Holding down details

In some cases, restraint must be provided against uplift at bearings against the effects of vehicle collision with the bridge soffit. The choice of suitable detail that does not compromise bearing function, inspection and maintenance requires careful consideration.

Structural configurations that result in uplift at bearings due to live load should be avoided.

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8 DESIGN FOR FATIGUE ENDURANCE

In this Section, any references to specific clauses are to clauses in BS 5400-10, unless noted otherwise.

8.1 General

Fatigue is frequently a more significant factor in the design of steel railway bridges than it is in highway structures. This is because live loads regularly approach the maximum design loading, because many steel railway bridges have short spans with a high live/dead load ratio and because floor members carry predominantly live load, subjected to a large number of axle loads per train. In such cases, the section sizes of members may be governed by fatigue stress limitations rather than by ultimate loads. Fatigue should be considered at an early stage during preliminary design.

Because of fatigue considerations (and also deflection limitations – see Section 2.2.4), it is rarely worth using a steel grade higher than S355 for railway bridges and in many cases grade S275 is appropriate. Generally, fatigue considerations tend to determine member sizes for spans under about 25 m.

Fatigue design is usually carried out to BS 5400-10, for which it is necessary to establish a standard traffic mix (heavy, medium or light) and annual tonnage to represent the traffic that it is anticipated the bridge will carry during its design life. This information should be provided by the railway authority. Guidance is given in GC/RC5510 Appendix D.

Fatigue failure of steel depends on the propagation of cracks in regions that are subject to fluctuating stress. The fatigue life depends on the size of the initial imperfection or stress concentration and on the range of the stress variation.

Part 10 allows for the size of imperfection or stress concentration by means of a comprehensive classification of welded and non-welded details (Table 17). The designer must identify the appropriate classification for the detail being considered.

Good detail design, avoiding features with poor fatigue classes and unpredictable secondary stresses, is more important than refinement of calculation. The actual fatigue life of the structure is much more sensitive to stresses at the critical details than to the number of cycles. Careful specification and supervision of fabrication is essential to ensure that the design assumptions are realised in practice.

8.2 Methods of assessment

The Code provides two methods for the assessment of fatigue life of railway bridges. The methods involve different determinations of the effective range of stress variation. In order of increasing complexity, they are:

- Without damage calculation – limiting stress range (Clause 9.2).
- With damage calculation – application of a loading spectrum (Clause 9.3).

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The first method is most commonly used and is much the quickest, though somewhat conservative. In some cases, where this method indicates failure by a small margin, designers may opt to recheck by the second method.

The procedure for the damage-calculation method is complex, although it is fully described in Part 10. That method would normally be used only on large and complex structures, or where a non-standard fatigue loading spectrum must be considered.

8.3 Fatigue loading

Fatigue loading is specified by Part 10 but essentially consists of the repeated application of proportions of the standard loading, RU or RL loading as appropriate. The standard loading, including the relevant dynamic factor and lateral loads, is applied to such parts of the bridge as necessary to determine the maximum and minimum stresses at the detail being considered. Where the passage of a train results in more than one stress cycle, the loading is also progressively applied across the bridge, to determine the magnitude of the individual stress ranges.

In the simple method (without damage calculation), only the full RU or RL loading is applied in this way.

In the damage calculation method, numerous proportions of standard loading are applied as a progressive load across the bridge, on each track. The spectra given in Part 10 have up to 7 components, so for a twin-track bridge, 14 loading transits must be considered; the procedure is therefore complex, although designers should be able to make use of customised spreadsheets to eliminate much of the tedium.

For light railways, a relevant loading spectrum must be determined, for use with the damage calculation method. Loading events on light railway structures can be very frequent and it is important not to underestimate the potential growth in traffic over the life of the bridge.

8.4 Classification of details

Fatigue detail classifications relate to the potential imperfections at welds, holes or other discontinuities, and their relationship to the direction of the stress variation. The greater the imperfection, the lower the stress range that can be tolerated for a given fatigue life. The complete range of classifications is shown in Table 17 of Part 10. This Table shows a range of typical details in classes A to G, plus W (for welds) and S (for shear connectors). For a given stress range, class A has the highest fatigue life, class W the lowest (class S is a special case).

The attachment of web stiffeners or other ‘short’ elements not carrying load in the stressed direction produces a Class F fatigue detail. Doubler plates and other ‘long’ welded attachments, such as bearing plates and patch plates introduce a Class G detail , as do other attachments that are close to the edge of the flange. Shear connectors produce a Class F detail in the stressed direction of the flange plate (as well as the Class S detail that requires special attention). Grip bolted splices introduce Class D details. Cross bracing, which introduces transverse restraint forces, involves a variety of details, including Class W.

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Because railway fatigue loading is onerous, particularly on members that experience a stress cycle for every axle, there is a strong incentive to use the highest categories of detail that are practicable. For example, for cross girders it is best from a fatigue point of view to use rolled sections without any fabricated attachments. However, there is a warning note to Table 17 that classifications better than Class D require a higher level of workmanship and inspection than can normally be achieved (i.e. when fabrication is to BS 5400-6).

To ensure that the workmanship will be appropriate to the fatigue life and detail class assumed by the designer, Part 10 recommends that all areas where class F or better is necessary should be marked on the drawings. The marking should show the fatigue class and the direction of stress to which it relates (see Clause 5.3.1). Workmanship levels, which are specified in Part 6, relate the level of inspection and the acceptance criteria for imperfections to the minimum fatigue class required by the designer.

Although deterioration due to fatigue, to a stage where failure occurs, is an ultimate limit state, as noted in clause 3.2 of Part 1, the partial factors γ_{fL} and γ_{f3} are both taken as 1.0, as stated in clause 7.1 of Part 10.

8.5 Limiting stress range

The simplified procedure for fatigue assessment involves the application of standard railway loading (on up to two tracks) to appropriate lengths of the point load influence line (for the position being considered) and the determination of the algebraic maximum and minimum values of stress. The stress range is then compared with a limiting stress range that depends on the non-propagating stress range for the class of detail and five modifying factors. The modifying factors allow for:

- Design life (where it is other than 120 years)
- The number of loading cycles from the passage of one train
- The type of loading (heavy, medium light traffic) and the base length of the load influence line
- The annual tonnage of traffic
- The ratio of the stress due to one track loaded to the stress with two tracks loaded

Tables and expressions are given in Part 10 for each of these modifying factors and for the value of the non-propagating stress range for the class of detail.

Note that the limiting stress range for unwelded reinforcing bars is given in Appendix B of GS/RC/5510 (this varies the recommendations in clause 4 of BS 5400-4).

8.6 Considerations for particular details

8.6.1 Doubler plates

Because railway bridge loading is relatively onerous and constraints on construction depth result in shallow steel girders, the use of doubler plates to achieve the necessary flange areas is common. However, the transfer of load

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into and out of the doubler plates at their ends produces a Class G fatigue detail for the main flange plate and a class W detail for the weld; the location of this termination and the detailing of the end of the doubler plate are primary considerations for the designer.

In a simple span, a doubler plate on the top flange may contribute nearly as much cross sectional area as the main flange plate; it is essential for strength over more than half of the span. But its point of termination is likely to depend more on where the Class G detail is acceptable for fatigue reasons than where its contribution to bending resistance is needed.

Guidance on detailing of doubler plates is given in Section 10.2.2, page 1.

8.6.2 Deck type bridges

As mentioned in Section 7.4.1, in multiple track deck-type bridges there is usually a pair of girders under each track, without any bracing between adjacent pairs of girders. If there were continuous transverse bracing, it would be subject to loading from every passage of a train. That load would have to be transferred through the webs of the main girders and would inevitably result in Class W fatigue details. It is better to avoid this problem by omitting bracing between adjacent pairs of girders.

With this structural arrangement, the transverse bending effects in the slab are usually fairly modest and the design of the slab is controlled mainly by practical considerations of slab depth and strength considerations at ULS and SLS.

At intermediate supports, stress ranges in the bottom flanges can be quite significant. The welding of a bearing plate to the underside of the flange introduces a class G detail and this imposes onerous limitations – the flange may need to be sized to keep the stress range within the limits.

8.6.3 Half through bridges

In half through bridges, the cross girders are subject to onerous fatigue loading (they effectively “see” every axle load) and, because of constraints on depth, are likely to be significantly stressed by that loading. Consequently, the beams must have the highest classification that is practicable; if rolled sections are used and welding is avoided (other than at the ends), Class C can be achieved in the tensile regions. On the top flange, Class F is inevitable where there are shear connectors, but the stress range should be lower. Where there is a steel deck, fatigue endurance will govern the design of both the Tee ribs and the deck plate.

At the ends of the cross girders, the necessity for U-frame action to stabilise the top flange results in stiff connections and poor fatigue class details. Careful analysis is necessary to determine the magnitude of the moments that are transferred and the connection can then be detailed accordingly. (Moments are developed due to F_c and F_R forces and to the differential deflection of the main girders, especially in skew bridges.) Note that welding patch plates to the web introduces a ‘long’ attachment (class G) and this may control the sizing of the flanges (in order to keep stress variations in the web within fatigue limits). If these ‘long’ attachments are welded to the top of the tension flange, this will also affect consideration of toughness (see Section 7.5.1). A continuous attachment to the web, such as the shear plates on the standard box girders, only introduces a class D detail.

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The 'standard box girder' configuration was developed with a virtually moment free connection between the cross girders and the boxes (the Tee rib deck units in Figure 4.13 are simply supported at the face of the web). The pinned connection is needed because the torsional stiffness of the boxes is so great that, with a more rigid connection, there would be such end restraint to the cross girders that the moments developed would give major problems for the fatigue design of the connection and of the web of the box section.

It should be noted that the use of steel deck on closely spaced cross girders results in more than a single load cycle on the cross girders (and the deck over them), due to the passage of a single RU train. Consequently, the value of the factor k_2 (one of the factors referred to in Section 8.5 above) is more accurately determined by use of the 'reservoir method' (that procedure is normally only used in the 'with damage calculation method'). The passage of the RU train results in four small stress ranges and one large stress range but the factor k_2 is typically only slightly less than unity when calculated by this method.

8.6.4 Filler beam deck bridges

Rolled sections are normally used in filler beam construction. The drilled holes through the webs of the beams introduce a Class D detail but this does not normally govern design.

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9 DESIGN FOR DEFORMATION PERFORMANCE

9.1 General

In comparison with road bridges, bridges supporting rail traffic are subject to more onerous deformation limits, to ensure the safety and comfort of rail traffic and passengers. Excessive bridge deformations can endanger traffic by creating unacceptable changes in vertical and horizontal track geometry, creating excessive stresses in rails and creating unacceptable vibrations in the bridge structure. Excessive deformations can also affect the loads imposed on the track/bridge system, and create conditions which cause passenger discomfort.

The relevant existing standards for deformation limits are GC/RT5112[7], GC/RC5510[5] and BD37[40], together with the referenced UIC Leaflet 776-3R[35]. It is likely that deformation requirements will be rationalised in the future, as Network Rail intends to adopt EN1990 Annex A2[31] (at the time of writing this Annex and the supporting National Annex are still under development). Also, the UIC has a current project to update the deformation requirements to reflect the studies carried out by the UIC in support of the development of Annex A2.

Generally, deformations are calculated using nominal loads and, if relevant, taking into account the deformation of substructures (most substructures are massive and have negligible influence on the total deformation at track level). The live load to be taken into account includes vertical loading enhanced by dynamic factors, centrifugal loads, nosing and longitudinal loads due to traction and braking. For ballasted decks, effects such as creep and settlement of foundations may be assumed to be addressed by track maintenance.

9.2 Deflection under permanent loads

The midspan vertical deflection of the bridge under dead and superimposed dead loads is checked to ensure that the natural frequency of the structure is within the known limits of validity of the allowances for dynamic load effects in the dynamic factors used in design. When determining the deflection, nominal loads should be used (i.e. take $\gamma_{FL} = 1.0$), except that an additional 100 mm of ballast depth should be taken into account (see GC/RC5510 Appendix E).

The limits on deflection under permanent loads are given in UIC 776-3R, but note that Appendix I GC/RC5510 gives a correction to the formula for the upper limit in Figure 1.

The vertical deflection under dead loads is also used to establish the required camber. See sections 2.4.2 and 6.5.6.

9.3 Deformation due to rail loading

When determining the live load deformation, all tracks should be considered loaded (if that is more onerous).

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9.3.1 Vertical deflection at midspan

Vertical deflection at midspan is checked to ensure acceptable vertical track radii, that the structure is not significantly different in performance to existing rail bridges and to ensure acceptable levels of vertical acceleration inside coaches corresponding to satisfactory passenger comfort.

Permitted span/deflection ratios as a function of structural configuration, span length, speed and passenger comfort are given in UIC Leaflet 776-3R. GC/RC5510 clause 19.8.2 gives advice on the selection of the appropriate level of passenger comfort and corresponding table from UIC 776-3R. Although the Tables in UIC 776-3R allow greater deflection for 'ordinary' lines with lower speed trains, an upper limit of span/600 should be observed for all Network Rail bridges (this requirement is already in the draft Annex A2[31]).

9.3.2 Track twist

Track twist under load occurs when, for a cross section normal to the track at any given position, the deflection of the structure under one rail is different from that of the other rail. Excessive track twist will cause a train to derail.

Track twist should always be checked on skew bridges. It can also occur, to a lesser extent, on bridges subject to eccentric live load, e.g. on a twin-track bridge with one track loaded.

Track twist should be considered for all locations, from off the bridge (no twist) through the transition region onto the bridge, across the bridge, and through the transition region off the bridge. Twist is checked along the centreline of each track over a gauge length of 3 m parallel to the tracks, taking into account the worst possible combination of tracks loaded and position of rail loading. For bridges carrying more than one track, identification of the critical loading pattern requires careful consideration.

The limiting values of track twist are given in UIC Leaflet 776-3R. These limits apply to the total twist at rail level. Where the track is on a transition between level and canted track, there is an intended rate of change of cant, which means that there is an intended twist in the track. The total track twist is the sum of twist due to structural deformation and any intended track twist.

Twist effects are likely to be particularly severe for highly skewed bridges, where track twist limitations may govern design, and in regions of skewed intermediate supports in a series of simply supported spans.

9.3.3 Uplift at bearings

Unrestrained uplift at any bearing is not permissible, to avoid the resultant vertical displacement of the track and to avoid premature failure of the bearing. In particular, uplift should be checked for bearings at the acute corners of highly skewed bridges and at the ends of continuous structures. See Sections 4.1 and 6.4.4.

9.3.4 Uplift at the ends of a deck

Uplift at the end of a deck, commonly known as "kick-up", occurs where the deck beneath the track extends (away from the span) beyond the bearings. The vertical deflection in the span causes a rotation about a transverse axis at the end of the deck and the small change of gradient at the bearing causes the end of the deck to deflect upward.

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Limits are given in UIC776-3R for uplift to avoid destabilising the ballast and limit uplift forces on track components and ensure acceptable additional stresses in the rails. Current thinking is that the limits in UIC 776-3R are too lax. To maintain acceptable track quality, recent European research indicates that the uplift should not exceed about 2 mm. This limit is likely to be included in a future Network Rail standard on the design of bridges.

9.3.5 Vertical acceleration of the deck

Special considerations apply to the design of rail bridges for speeds over 200 km/h, to guard against excessive vibrations. Excessive vibrations can lead to ballast instability and unacceptable reduction in wheel rail contact forces. The design of these bridges is outside the scope of this guidance and specialist advice should be sought from the railway authority.

9.4 Deformation due to rail loading and temperature

9.4.1 Rotation of the ends of the deck about a transverse axis

The rotation at the end of a deck, or the relative rotation between adjacent deck ends (where there is a series of separate spans) must be limited, to limit the uplift forces on track components, to ensure acceptable additional stresses in the rails and to limit angular discontinuities in rail expansion devices and at switches.

The limits are given in UIC Leaflet 776-3R. Where track/bridge interaction effects are required to be taken into account (see below), the associated checks on limiting additional rail stresses may be critical.

For decks with non-ballasted track, the effect of rotation of the end of the deck and any uplift at the end of the deck need to be taken into account when checking the load effects on the rail fastenings and compared with the relevant limit state (including fatigue) performance characteristics of the rail supports and fastening system.

9.4.2 Track/bridge interaction

Vertical deflection in the span causes a rotation about a transverse axis at the end of the deck and, depending upon the height of the upper surface of the deck above the bearing, a corresponding longitudinal displacement. Together with longitudinal displacement of the substructure, due to traction and braking loads and temperature contraction and expansion, these actions develop additional stresses in the rails and additional forces on the bearings. This combined response of the bridge to variable actions is called track/bridge interaction and can dictate the design of long span bridges or continuous bridges.

Where track/bridge interaction checks are required to be carried out by GC/RC5510 (clause 12.1), clause 6.5.4 of BS EN1991-2[4] should be assumed to supersede the advice in UIC Leaflet 774-3R[37].

9.4.3 Longitudinal displacement of the end of the deck

Unless track/bridge interaction effects are required to be taken into account by GC/RC5510 clause 12.1, there are no specific requirements to be met.

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9.5 Lateral deformation

The total lateral deformation of the deck under variable loads (centrifugal load, nosing, wind load, lateral temperature gradient) should be limited, to ensure acceptable track geometry and passenger comfort.

Limits are given in UIC Leaflet 776-3R. The limits are defined in terms of the maximum permitted change in track radius and the maximum change of angle at the end of a deck. The maximum change of angle is about a vertical axis and should be assumed to apply to both ends of a deck and to the maximum total change of angle between adjacent decks.

To avoid the occurrence of resonance between the lateral motion of vehicles on their suspension and the bridge, the following limits must be observed:

- The lateral flexibility of the bridge should not exceed the limit in GC/RC5510 clause 19.8.3.
- The lateral frequency under permanent loads should not be less than the limit in GC/RC5510 clause 19.8.3.

The above limits are not likely to be critical for short to medium span bridges with solid decks and high in-plane shear stiffness.

9.6 Effect of deformation on clearances

When checking clearances, allowance should be made for deflection of the structure. In some situations the requirement to maintain clearances within a limited range can dictate the design. For example, variations in vertical clearance and stepping distances between a train and a platform can dictate the design of an underline rail bridge at a station where the platform is supported on a separate structure and does not 'move with the bridge' under rail loading. Specialist advice should be sought in such cases.

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10 BRIDGE DETAILS

This Section presents a series of details for the various forms of construction presented in Section 4 and for general aspects common to several forms of construction. Comments are given in relation to the following considerations:

- Fatigue endurance
- Fabrication
- Installation
- Durability

For any detail, consideration should also be given to ensuring ease of inspection and maintenance for all elements and connections.

All details should be checked for adequacy of access for welding, bolting, inspection and application of protective treatments

Details that are optimum in relation to one consideration may well not be optimum in relation to other considerations. Alternative details are presented where appropriate.

10.1 Shallow deck-type bridges

10.1.1 Filler beam deck

Filler beam decks can be constructed with either UC or UB sections (or the corresponding HD and HE European sections). The use of UC or HD sections results in a shallower depth and the spacing between beams is usually less than the flange width. A typical section is shown in Figure 10.1. Holes drilled in the beam webs should be at lest 15 mm greater than the size of the rebars.

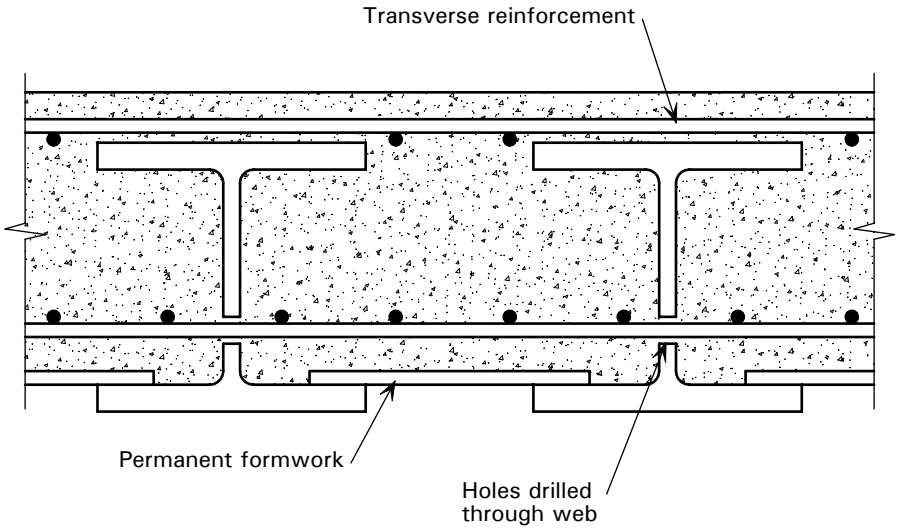

Figure 10.1 *Cross section of filler beam deck*

Since there is no welding to the rolled section, only the drilling of the webs, the fatigue classification is very good (class C generally, class D at the holes). Fabrication work is minimal, although for longer spans the rolled sections will need to be precambered (Network Rail requires precambering for spans over 12 m).

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Where the deck cannot be precast to full width (permitting erection by lifting in the complete deck), longitudinal joints will be needed between the partial width units. This may have to be an in situ infill between the sections either side of the joint and with wider gaps between UB or HE sections the reinforcement may need to be arranged with overlapping links to provide a shear key.

10.2 Half through plate girder bridges

10.2.1 General

The key structural components of a half through plate girder bridge are shown in Figure 10.2. This shows a skew configuration, which is common. The configuration is applicable to bridges with composite decks, filler beam decks and all-steel decks. For illustrative purposes, support arrangements for the trimmers are different at each end and rocker bearings are shown at one end only (it is common practice for short spans to use line rockers at both ends).

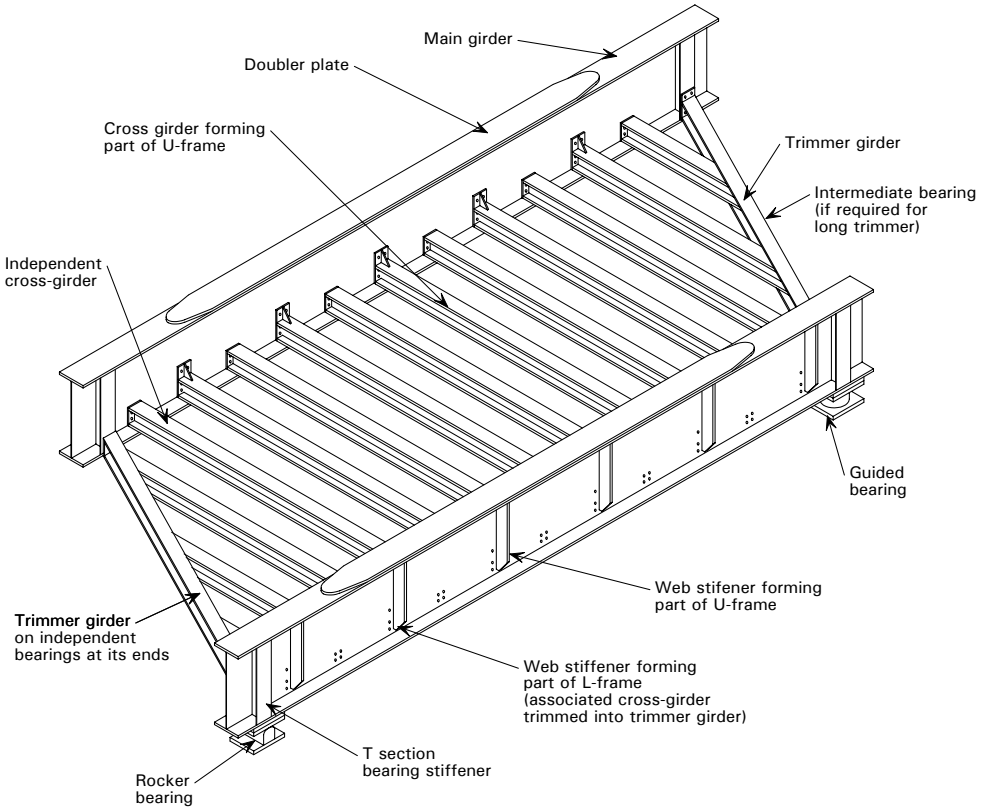
Main girder
Doubler plate
Cross girder forming part of U-frame
Trimmer girder
Intermediate bearing (if required for long trimmer)
Independent cross-girder
Guided bearing
Web stifener forming part of U-frame
Trimmer girder on independent bearings at its ends
Web stiffener forming part of L-frame (associated cross-girder trimmed into trimmer girder)
T section bearing stiffener
Rocker bearing

Figure 10.2 *Components of half through plate girder bridge (bridge floor not shown)*

10.2.2 Features common to most half through plate girder bridges

Web stiffeners

The intermediate stiffeners are on the outside of the web (to avoid vulnerability in the event of a derailment and to avoid awkward waterproofing details) and are usually simple flats, welded to the web and the top flange. The stiffeners are not attached to the bottom flange. See Figure 10.3. If the top flange outstand is the same width or narrower than the stiffener, the weld is kept clear of the edge of the flange by using a quadrant cut-out (minimum radius 40 mm).

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A cope hole is not necessary at the top flange/web junction; fabricators can grind the flat locally to miss the continuous longitudinal weld, before the stiffener is welded to the main girder.

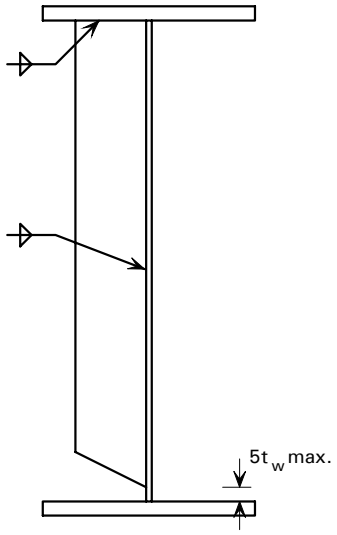

Figure 10.3 *Typical web stiffener for half through plate girder*

This provides a simple and economical fabrication detail. All welds are fillet welds and there is no fitting to the web or flange. Using the minimum size of weld for strength minimises the distortion of the web during welding. The fabricator should be allowed the option of laying the weld to the lower portion of the stiffener after the end plate of the cross girder is bolted to the web, to facilitate fit-up to the end plate.

The detail introduces a class F fatigue detail in the web and top flange. Avoiding the attachment to the bottom flange allows the bottom flange to be class D, if there are no discontinuities in the web/flange weld. Application and maintenance of protective coatings is straightforward.

Stiffeners will probably not be required at every cross girder position but should be aligned with the cross girders where they do coexist (and thus form U-frames).

Where stiffeners need to be welded to the bottom flange as well, for example to help transmit large moments from cross girders or to transmit restraint forces from lateral bracing, fillet welds should be sufficient.

Bearing stiffeners

With I-section girders, Tee-section stiffeners, one either side of the web, are usually sufficient to transmit the bearing reactions, except where the design thermal movement (from mean position) exceeds about 30 mm, equivalent to a length from the fixed bearing of about 50 m. See Figure 10.4(a). Boxed stiffeners have been used but these can give rise to fabrication/assembly problems and do not provide significant structural benefit, except where the torsional stiffness can be used to limit the effective length of the main girder in a tight clearance situation, such as when located in the ‘six-foot’ space. Open stiffeners are therefore preferred. On the inner face of the web, Tees are less vulnerable than flats in the event of a derailment and are substantially protected by an RC haunch (such as that shown in Figure 10.20), where one is provided.

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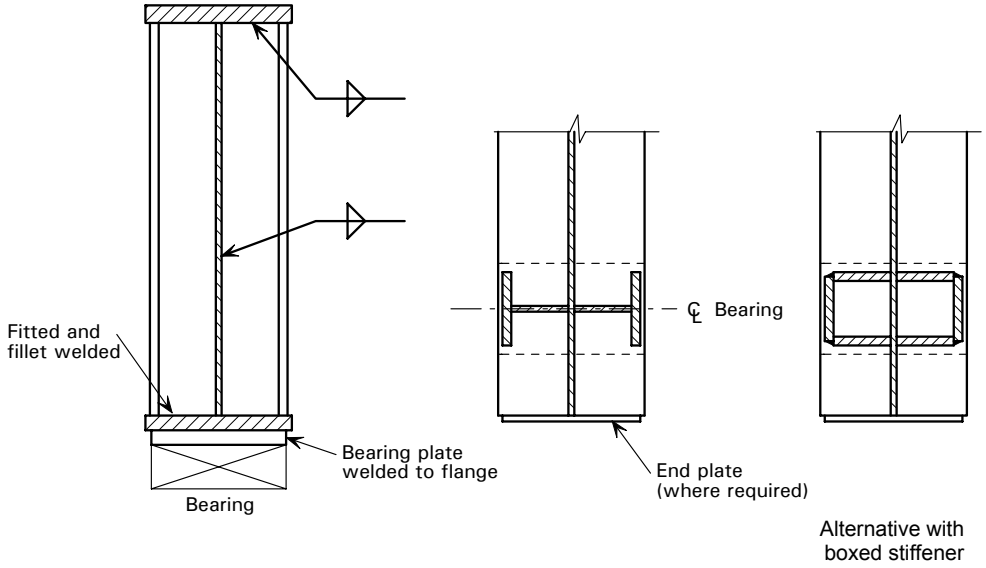
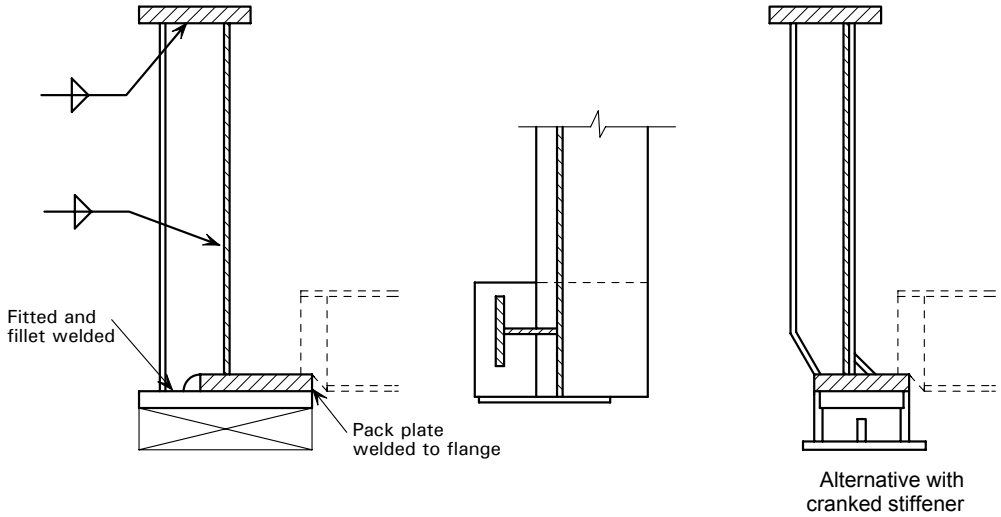
Open section bearing stiffeners should be fitted to the bottom flange and then fillet welded all round. This allows much of the ULS reaction to be transmitted in direct bearing. However, direct bearing cannot be relied upon to transmit fatigue load effects (see general advice in GN 2.04[47]) and fillet welds will be sized to suit fatigue limitations. Alternatively, full penetration butt welds can be specified at the bottom flange although this can lead to distortion of the flange, particularly with one-sided stiffeners. With box section bearing stiffeners, fitting is much more difficult to achieve and should not be assumed; full penetration welds at the bottom are also very difficult to achieve.

With Z-section main girders, an asymmetric arrangement must be adopted. The detailing of the stiffeners depends greatly on whether the trimmer girder is connected to the stiffener or whether it is on independent bearings. Examples where trimmers are simply supported on the bottom flange are shown in Figure 10.4(b). The alternative shown in that figure is a common detail but it is difficult to control the squareness of the flange to the web and its flatness, because of the asymmetric welding along the flange.



*a) Symmetrical arrangement with **I**-section main girder*



b) Arrangement with Z-section main girder

Figure 10.4 *Bearing stiffeners in half through plate girder bridge*

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Doubler plates

Because of limitations on thickness of flange plates to meet toughness requirements (see Section 7.5.1), the flanges are sometimes fabricated by welding doubler plates to the primary flange plates. These doubler plates are often shorter than the primary flange plates, in recognition of the reducing moments away from midspan. Where the doublers are terminated in the span, a detail such as that shown in Figure 10.5 should be used.

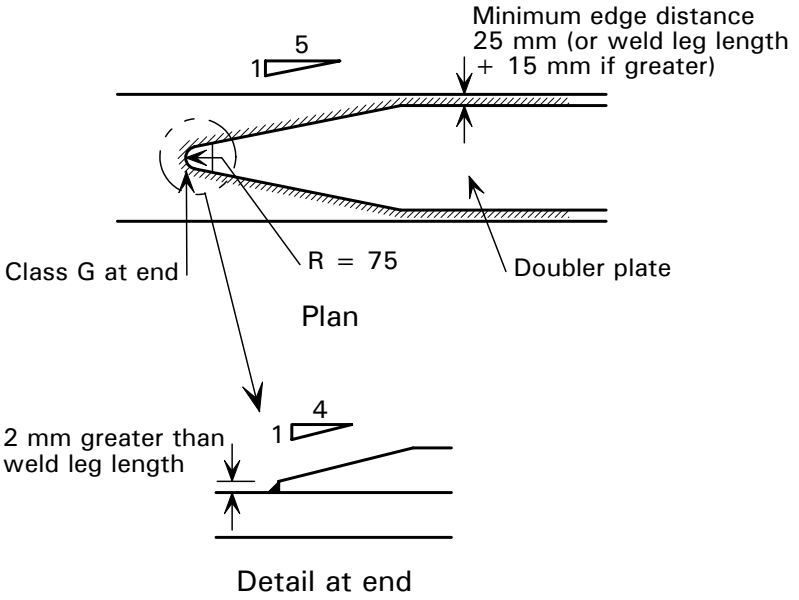

Figure 10.5 *Doubler plate detail at termination*

The taper in width and thickness are both needed to ensure a smooth flow of load between doubler and primary flange and the radius helps to minimise the potential for fatigue cracking. Nevertheless, the termination introduces a class G detail into the primary flange plate and the shear through the weld is a class W detail.

The termination detail is also a potential site for the initiation of brittle fracture and the termination of doubler plates in tensile regions must be considered very carefully (see discussion in Section 7.5.1).

If thicker flange plates are acceptable, rather than doublers, the fatigue classifications would be better, even where the plate is spliced with a full strength butt weld.

Trimmer girders

Trimmer girders support the end of the deck and may act as part of end U-frames or as part of a bracing system at an end support. On a skew half through bridge the trimmers also provide end support to trimmed cross girders that are curtailed because of the skew. Typical details of trimmer to a steel deck and the connection of a trimmed cross girder are shown in Figure 10.6. The Figure also shows the provision of a concrete end face to the trimmer - the use of concrete (with drip feature) to protect the external face of the trimmer beam and provide the transition interface between the deck and the ballast wall (see also Section 10.6) is desirable because of the difficult access for maintenance.

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Trimmers for two-track half through bridges are usually deeper than the intermediate cross girders - the extra depth extends below the bottom of the main girder and the ends may be tapered to the same depth as the intermediate cross girders. Where there is also a significant skew, an intermediate bearing (at the midspan of the trimmer) may be provided to limit deflection.

For details of the connection of trimmers to the main girders in half through bridges, see Figure 10.16. Where no moment connection is provided between the trimmer and the main girder, the trimmer may sit simply on the bottom flange (as shown in Figure 10.4) or be supported completely independently of the main girders.

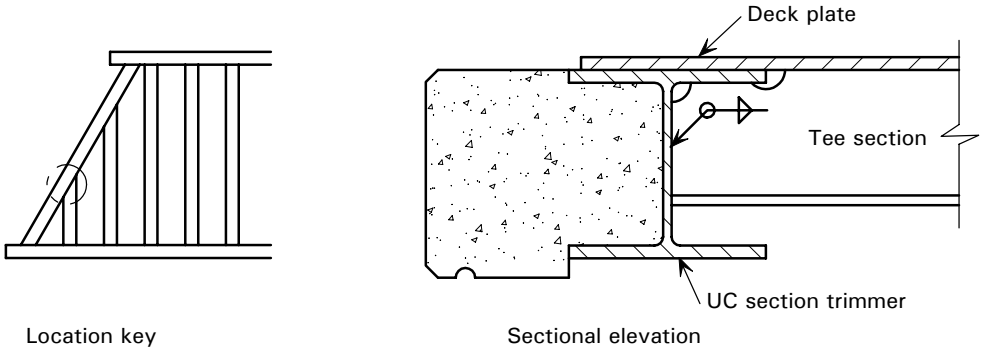

Figure 10.6 *Connection of trimmer and trimmed cross girders on a skew bridge with steel deck*

Where composite decks are used, connection of cross girders to trimmers of skew bridges may be made using an end plate detail, as shown in Figure 10.7. A notched end to the cross girder is simpler than a tapered detail but fatigue at the re-entrant corner needs to be checked. A minimum radius to the corner must be specified and the cut should be machined or ground, if necessary, to ensure a smooth profile.

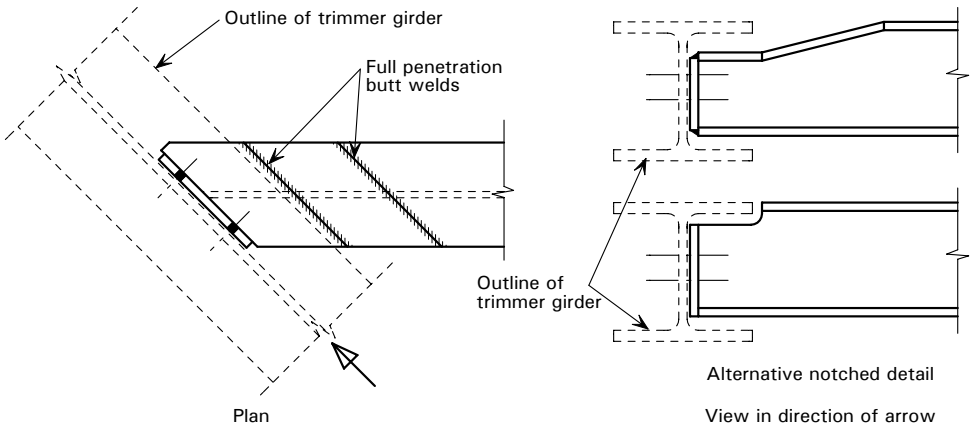

Figure 10.7 *Bolted connection of trimmed cross girder to skew trimmer*

Fanned cross girders

Cross girders are normally arranged square to the main girders. Where there is significant skew, they are usually trimmed at the ends of the span, as shown in Figure 10.6. With a small skew (up to about 25°), the endmost cross girders are often detailed in a 'fanned' arrangement, between the skew trimmer and the square arrangement in the middle of the span, to avoid trimmed cross girders meeting trimmers at a very acute angle (see Figure 10.8).

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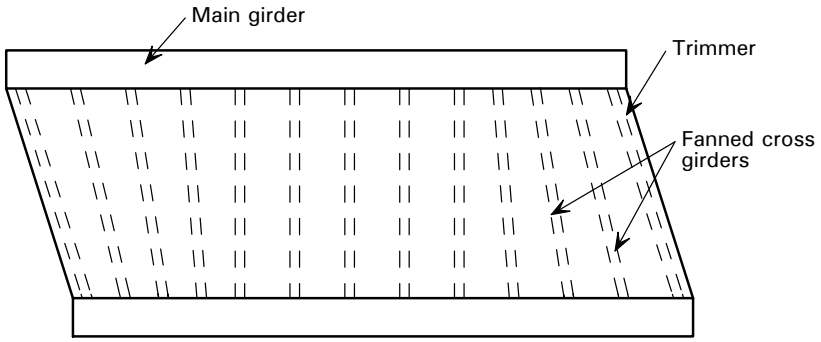

Figure 10.8 *Arrangement of fanned cross girders*

10.2.3 Half through bridges with composite deck

The following discussion and details relate to decks where either:

- the cross girders are part of a filler beam deck (encased, but with flanges exposed, similar to Figure 10.1 but with more widely spaced beams), or
- the slab is haunched around the cross girders (such as in Figure 4.12), or
- the slab is on top of the cross girders.

Cross girder connections

Bolted stiffened end plate connections to the face of the web are usually sufficient to transfer the end moments from the cross girders (due to F_c and F_R forces).

There are a number of details that are commonly used for end plate connections, some of which are illustrated below.

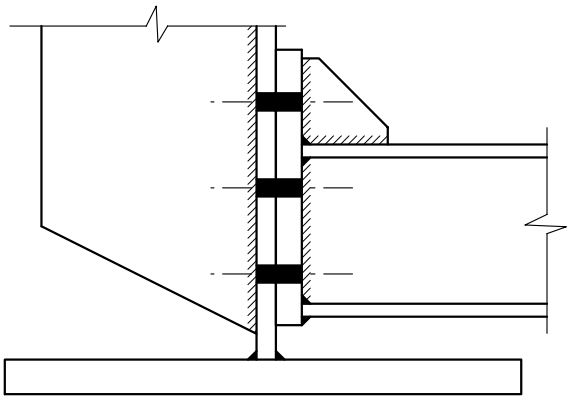

Figure 10.9 *Six-bolt stiffened connection to shallow cross girder in a single-track bridge*

The arrangement shown in Figure 10.9 is suitable at web stiffener locations (where U-frames are created) with shallow cross girders (160 mm to 225 mm deep) at close centres (600 mm) carrying a single track. The detail shows a clear gap, ideally of 50 mm, between the cross girder soffit and the top of the bottom flange; this avoids corrosion traps and allows access for maintenance of protective coatings etc. although this does add to the overall structure depth. Where the deck is filler beam construction, the gap below the cross girder is usually filled and the slab extended down locally to the flange. Bolt heads

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should normally be on the outer face, especially when the endplate is encased in the concrete of the deck slab (so that any future failure can be detected).

Where the web is thin (less than 20 mm), a patch plate may need to be welded to the web to cater for local bending due to the tensile forces transferred by the bolts; the plate also helps to ensure flatness for fit-up. See Figure 10.10. This patch plate introduces a class G detail on the web and is thus a lower class than due to the attachment of the stiffeners. (Note that attachment to the flange should be always be avoided, since the class G detail would introduce onerous requirements for toughness of thick flanges in tension – see discussion in Section 7.5.1.) Alternatively, to avoid the use of a patch plate, a thicker web, than required for shear, may be used; a 20 mm web was sufficient for the 1996 Z type single-track bridge.

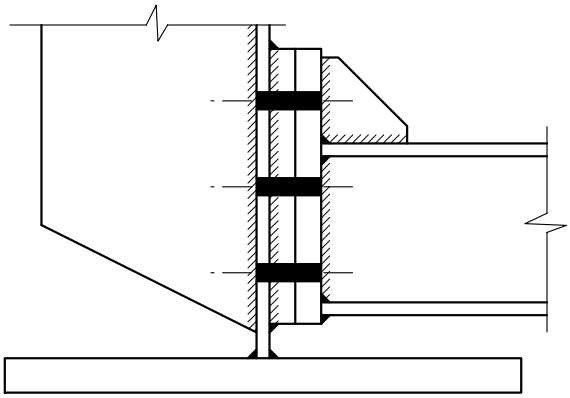

Figure 10.10 *Six-bolt connection to a thin web*

Connections of cross girders for twin tracks and for wider cross girder spacings may be made using a similar detail to that in Figure 10.10 but the cross girders will be deeper (up to about 400 mm). More bolts will be required and a larger stiffener will be needed.

With both these details, shear is transferred in friction at SLS (and for fatigue design) but the bolts can be assumed to slip into bearing/shear at ULS. The welds around the end of the cross girder must be designed to transmit both shear and tension/compression and must be checked as a class W fatigue detail. The details in Figure 10.9 and Figure 10.10 can usually be assumed to achieve a flexibility parameter f (clause 3/9.6.4.1.3) of 0.2×10^{-10} rad/N mm.

For the same bridge configuration, cross girders between web stiffener locations would be connected as shown in Figure 10.11. A four-bolt connection is sufficient in most cases, for closely spaced single-track cross girders.

In each case, welding the end plate to the cross girder potentially distorts the end plate (so that its surface is no longer plane) and the fabricator may choose to machine the plate after welding; a slightly thicker plate (than required by the designer for strength) will be provided to allow for this machining. End plates thinner than 25 mm should not be specified.

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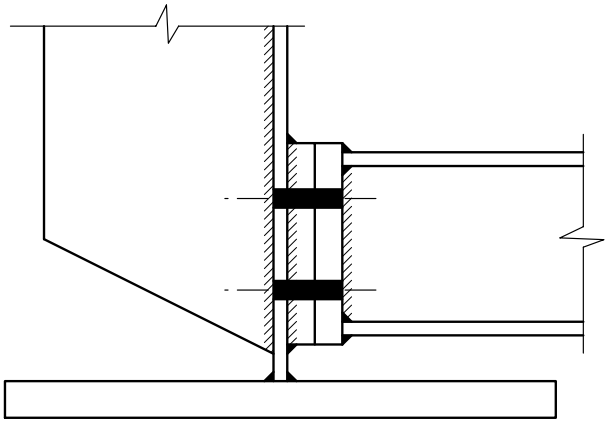

Figure 10.11 *Four-bolt connection of single-track cross girder that is not part of a U-frame*

In twin-track bridges, the combination of vertical and longitudinal shear is likely to be too great for the small number of bolts within the depth of the cross girder. In such cases a 'shear plate' detail, such as shown in Figure 10.12 may be used; vertical shear is carried in bearing, longitudinal shear is carried in friction. This also facilitates erection of cross girders and provides redundancy against a cross girder dropping in the unlikely event of premature bolt failure.

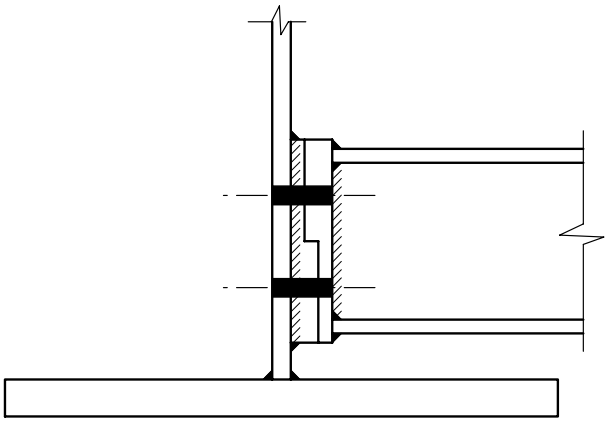

Figure 10.12 *Four-bolt connection using a shear plate*

Where the structure depth is tightly constrained, the cross girders may sit on and be connected to the bottom flange, as shown in Figure 10.13. Bolt spacing and edge distances should comply with the usual rules in Part 3. To minimise construction depth , the heads of the bolts should be countersunk in the bottom flange. The bolts must then be tightened from above – with shallow cross girders, it may be necessary to reduce the width of the top flange locally, to allow access for the tightening wrench from above the deck. With this detail the flexibility parameter f (clause 3/9.6.4.1.3) may be taken as 0.1×10^{-10} rad/N mm.

In addition to the cross girder end plate to main girder web bolts being designed to resist vertical shear, it is good practice to design the main girder web to bottom flange weld also to resist the local effects of the cross girder reaction. This provides redundancy to the connection and avoids any risk of overloading the main girder bottom flange to web weld (one side of which is permanently hidden from inspection).

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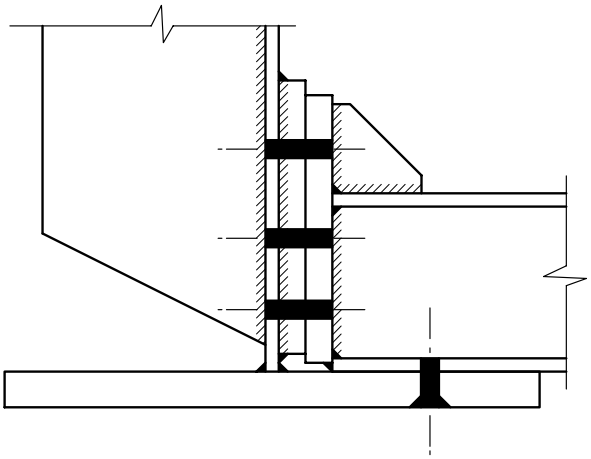

Figure 10.13 *End plate connection bolted to main girder*

Instead of flat endplate details, such as illustrated above, bolted 'shear plate' details, similar to that in Figure 10.12, have been used at U-frame connections, to speed erection and to reduce the number of bolts, but it is a more costly detail and can give fit-up problems.

Swan neck cross girders

Where construction depth is very tightly constrained, it may be necessary to use 'swan neck' cross girders, similar to that shown in Figure 10.14. The soffit of the cross girder can then be at the same level as the soffit of the main girder and the depth of the cross girder is minimised by using a heavy bottom flange and by designing for composite action with the deck slab (which can be achieved through studs welded on the web of the cross girder or by passing longitudinal bars through the web).

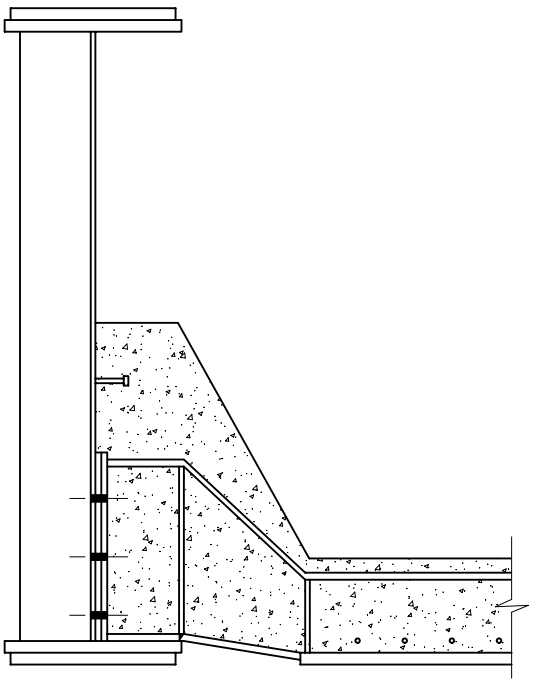

Figure 10.14 *'Swan-neck' cross girder*

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Typical connection of cross girders in London Underground half through bridges supporting single tracks

For London Underground half through bridges, web stiffeners are provided on the inside faces of the webs, protected by reinforced concrete haunches. Flats are used for the web stiffeners and these stiffeners are terminated at horizontal endplates, as shown in Figure 10.15. This detail facilitates both vertical and horizontal bolting of the cross girders to the main girders. After all bolting is completed, the cross girders are encased in concrete and the haunches cast integrally with the deck slab.

This connection achieves a flexibility parameter f of 0.2×10^{-10} rad/Nmm.

For double track bridges on London Underground, limitations on construction depth usually require three main girders; similar connection details are used.

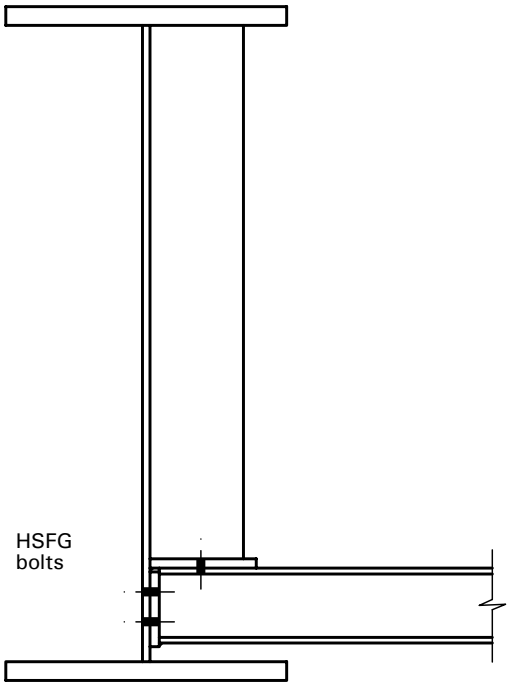

Figure 10.15 *U-frame connection on London Underground bridges*

Connection of end trimmers to main girders

If trimmers are expected to provide the torsional restraint to the ends of the girders, they will require moment connection to the main girders via the bearing stiffener.

If torsional restraint is provided to the main girders by a line rocker bearing, either the trimmer should not have a moment connection to the main girder (use a pinned support on the bottom flange or support the trimmer independently of the main girders) or, if a rigid connection is used, the rocker bearing must be designed for the end moment from the trimmer girder.

Examples of trimmer connections are shown diagrammatically in plan in Figure 10.16.

With a Z-section main girder, a pinned support for the end trimmer can be provided on the inner ends of line rocker bearings, as shown in outline in Figure 10.4(b).

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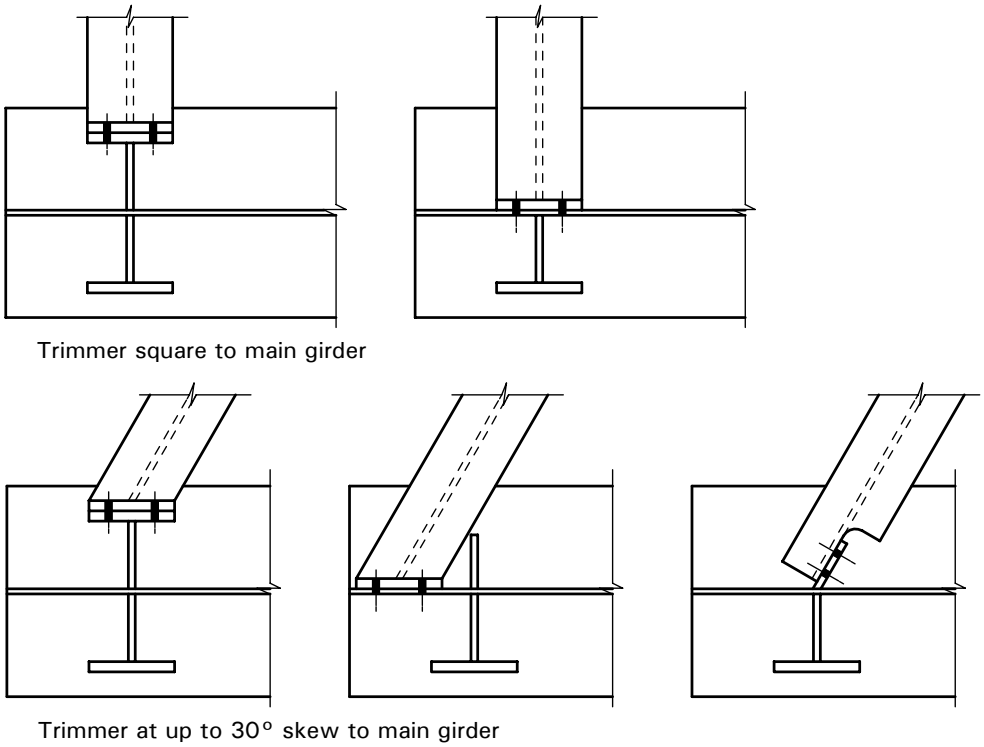

Figure 10.16 *Connection of trimmers to main girder (plan views) where bearings allow lateral rotation*

10.2.4 Half through plate girder bridge with battledeck floor

Figure 10.17 presents an isometric detail of the connection of a steel battledeck floor to an I-section main girder in a half through bridge. This form of construction is discussed in Section 4.3.6.

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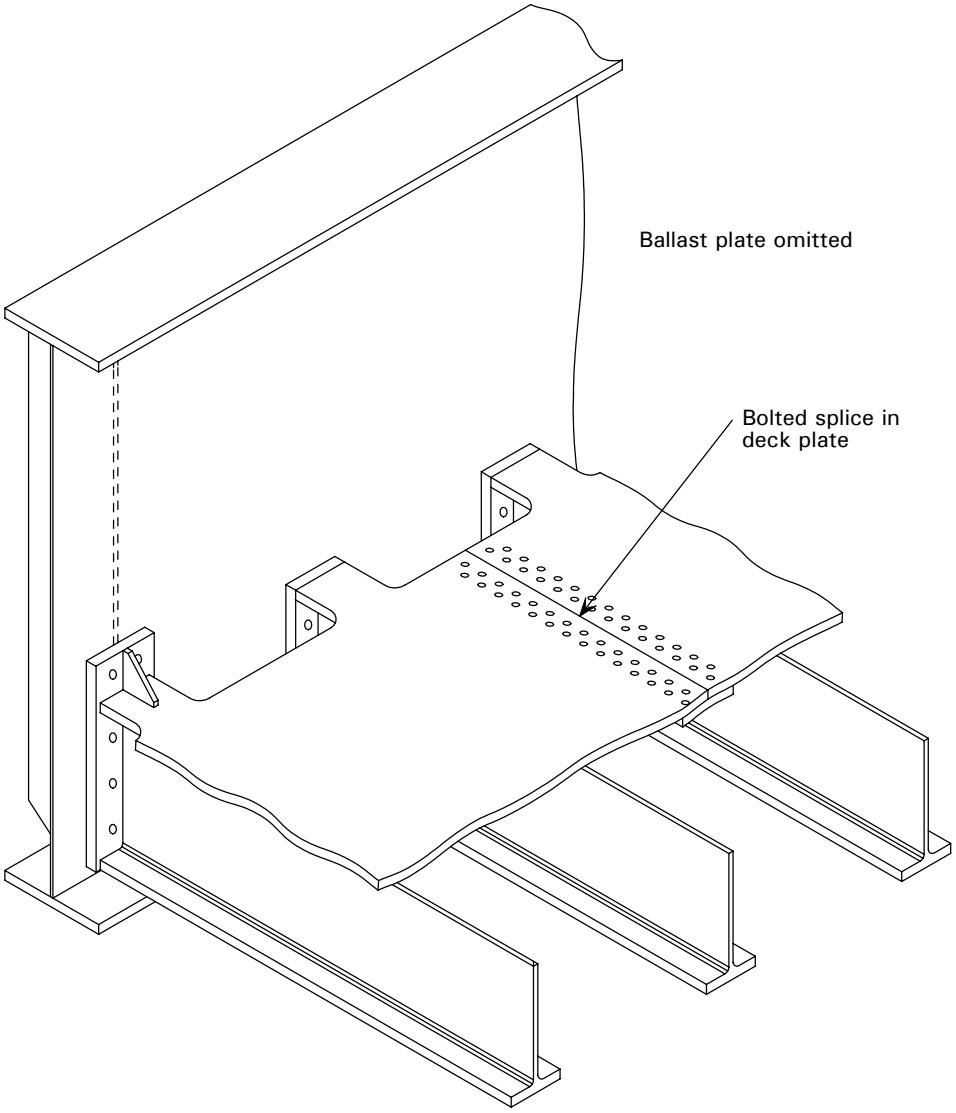

Figure 10.17 *Arrangement of battledeck floor and main girder*

Connection details at the end of the cross girders are shown in Figure 10.18. The triangular stiffener is sometimes elongated to provide room for a lifting eye for the deck panel.

Comments on the various components are as follows:

End plate to cross girder connection

Shear and moment from the cross girder are transmitted through an end plate that is friction grip bolted to the web.

The thickness of the end plate to the cross girder should not be less than 25 mm, to minimise the distortion during welding and bending under tensile loading on the upper bolts. Patch plates (welded on the face of the web) will only be required if the web is thin, 16 mm thick or less. Weld sizes should be kept to the minimum compatible with strength and fatigue endurance, to minimise distortion to the end plate. As for cross girders for composite decks, the fabricator will probably machine the end plates and this would normally be done after the Tees are welded to the deck plate, to ensure that the cross girders are to a common length and the faces of the end plates are in a common plane.

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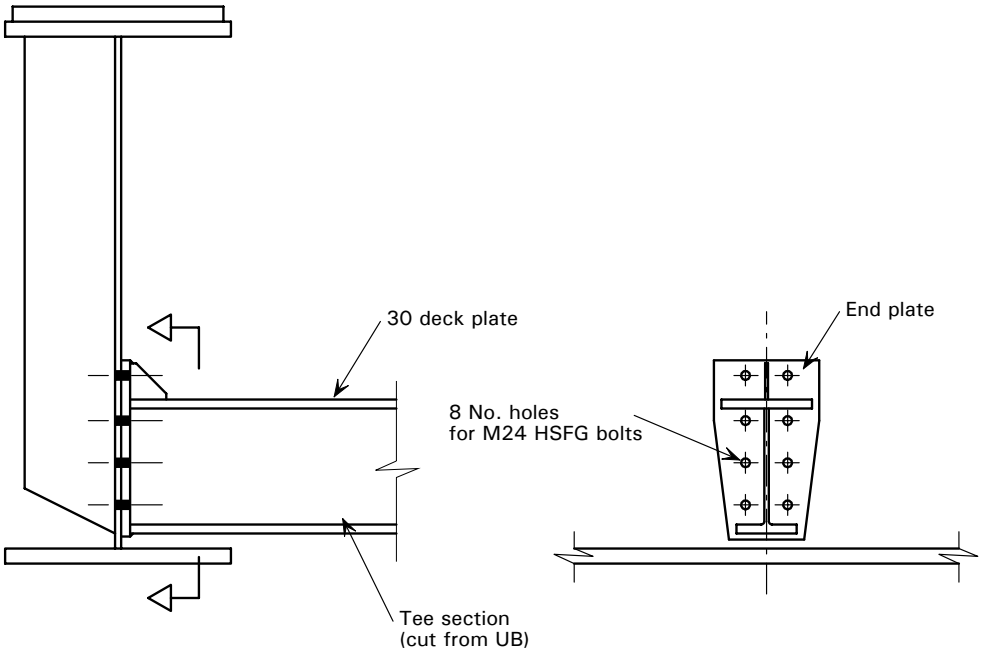

Figure 10.18 *Connection of battledeck cross girders to main girder*

As explained above, allowing a gap between the bottom flange of the cross girder and the top surface of bottom flange of the plate girder simplifies the connection detail. The gap should be sufficient to allow all surfaces to be maintained (at least 50 mm is desirable).

The HSFG bolts are designed against slip at SLS and under fatigue loading, and the welds between the end plate and the cross girder will probably be designed by fatigue considerations (Class W detail).

Deck plate

Between cross girders, the deck plate is kept well clear of the main girder web, for access during installation and maintenance. The internal corners of the plate should be rounded, minimum radius 75 mm or three times the plate thickness.

Splices in the deck plate are usually bolted, if the panels have to be placed during a possession. Welded splices can be used if the bridge is preassembled at site (before lifting or rolling/sliding into position). Welded splices should be at least 100 mm from the nearest Tee, to keep the weld away from the greatest fatigue stress range; the site-bolted splices should be located midway between the Tees, to maximise access for tightening. Splices must be able to transmit local shears due to derailment loading, so bolted splices have two rows of bolts on either side of the joint.

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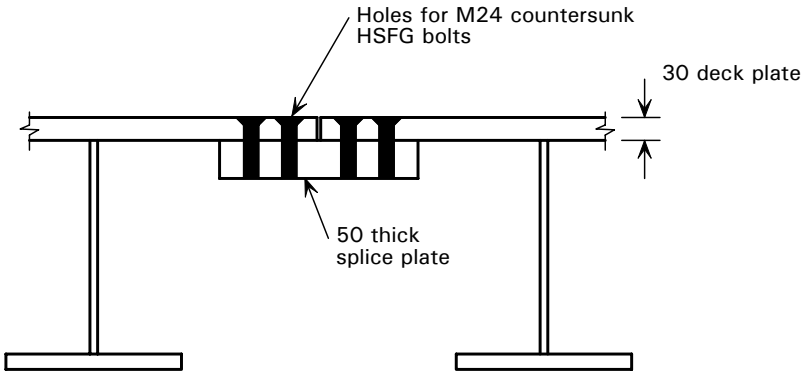

Figure 10.19 *Bolted splice in deck plate*

Where high strength clamp-type fasteners are used, because they are more quickly installed in the limited possession period, the splice must be offset from the midway position, with the fasteners on one side located centrally between the Tees. The fasteners on the side closer to the Tee are installed before the Tee is welded to the plate; the fasteners on the other side are installed on site. This arrangement allows space for the larger tightening tools for clamp-type fasteners.

The fabricator should be allowed the opportunity to vary the length of the deck panels to suit erection requirements and fabrication facilities (for example for machining end plates to a common plane)

Alternative detail

Where construction depth is at a premium, the cross girder is placed immediately on top of the bottom flange, as shown in Figure 10.13. When this arrangement is chosen, the cross girder bottom flange is preferably bolted to the bottom flange of the main girder. The edge spacing of the bolts should comply with the Part 3 rules to prevent interface corrosion.

10.2.5 Waterproofing and ballast retention

In half through bridges, the ballast must effectively be retained in a 'trough' and the waterproofing must extend up the sides of the trough. It is best to avoid using the structural elements as the sides of the trough: with a concrete floor, the slab can be returned up the face of the web (as 'haunches'); with a steel battledeck, a steel 'ballast plate' is used.

RC haunch

Where the slab is returned up the face of the web, it should be keyed to the web with shear connectors. The keying is needed to avoid separation, rather than to generate composite action; 19 mm diameter studs should be adequate for this purpose. At the top of the wall, a 'weather flat' should be provided, to avoid interface corrosion of the structural web. See Figure 10.20. The waterproofing to the deck extends up the face of the haunch as far as the weather flat. The protective coating system should extend over the full face of the web, to reduce any risk of corrosion. A sealant compatible with the waterproofing should be applied between the top of the concrete and the weather strip, and to any construction joint forming a kicker.

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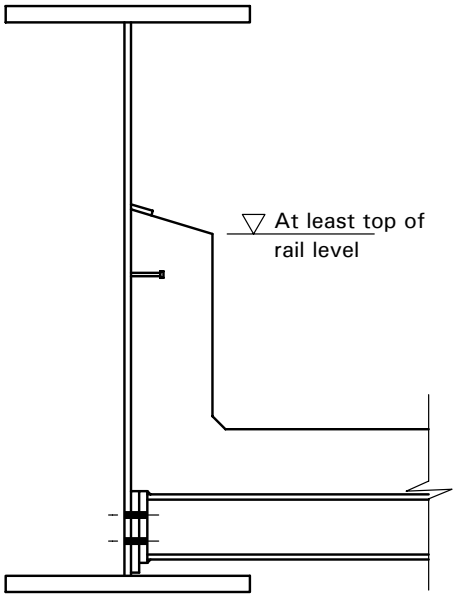

Figure 10.20 *Arrangement of RC haunch*

Note that this detail is not suitable for Z-section main girders, because that configuration will have been chosen to minimise the width between the webs of the main girders. On Z type decks the web face is usually protected by concrete slabs, up to the underside of the top flange.

Ballast plate

With steel decks, a non-structural ballast plate is provided to retain the ballast and keep both the ballast and any drainage away from the structural connections.

A common and simple arrangement for a ballast plate on a battledeck, is shown in Figure 10.21. The ballast plate is fabricated in short sections that can each be manhandled. The sections are bolted together through 'flanges' at their ends and these also provide stiffening against the pressure from the ballast. The plates may simply lie against the web or may be fastened near the top edge.

However, this arrangement has the disadvantage that access to the structural connections is only possible if the ballast is removed

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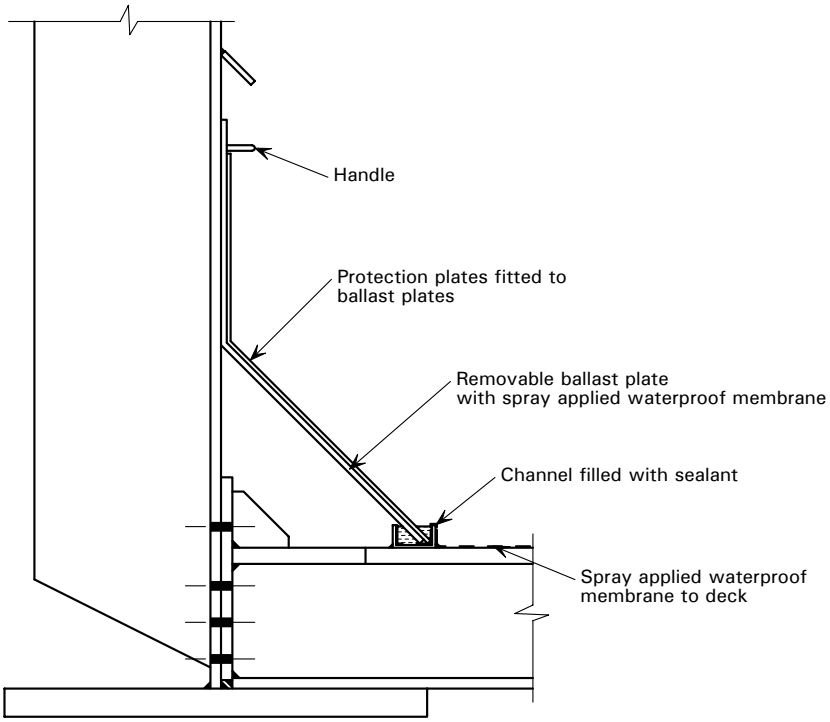
Handle
Protection plates fitted to ballast plates
Removable ballast plate with spray applied waterproof membrane
Channel filled with sealant
Spray applied waterproof membrane to deck

Figure 10.21 *Ballast plate (sloping face)*

An arrangement that allows ready access for inspection of the connections (albeit during possessions) would be as shown in Figure 10.22. If the portion of ballast plate above the top of the ballast is within the 'overhang' of the flange from the web, it does not need to be designed as a robust kerb.

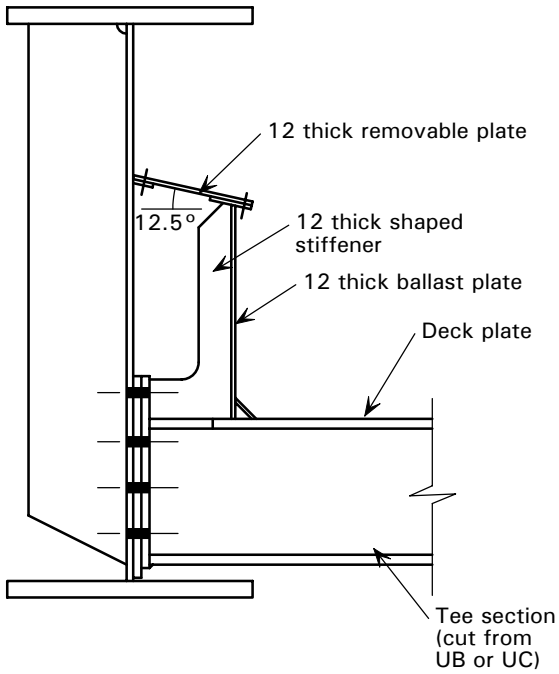

Figure 10.22 *Ballast plate (vertical face)*

Waterproofing

It is common practice and generally more economical to specify spray applied waterproofing systems, which, for steel decks, are usually applied in the fabrication works (except locally at site joints). However, where all the

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waterproofing system has to be applied on site, during a possession (and thus time is likely to be at a premium), sheet waterproofing membrane systems may be an alternative.

Compatible waterproofing membrane and mastic sealants should be specified, preferably from the same manufacturer, to avoid guarantee / warranty issues. Corrosion protection coatings to steel need to be compatible with the waterproofing system.

Additional protective boarding is usually fitted to give protection to the waterproofing membrane on the surface of the ballast plates and concrete haunches. This boarding affords additional protection to the membrane during routine track maintenance and hand or mechanical packing of the permanent way. The concrete is usually rebated to accommodate the boarding and to define its extent.

10.3 Standard box girder bridges

All the principal details for the standard box girder bridges are given in the Network Rail drawings and these should be used where standard box girders are required.

10.4 Slab-on-beam composite girder bridges

Details for slab on plate girder railway bridges are generally similar to those used in highway bridges, except that section sizes are likely to be greater and that more attention needs to be paid to the avoidance of lower class fatigue details. General guidance on a range of details is available in the Guidance Notes[47].

Plan bracing to the bottom flanges is not usually required, unless there is a need to improve torsional stiffness, for example when the span is long and a single track is eccentric to the pair of girders supporting it or if there is a risk of vehicle impact from below. Plan bracing would create a pseudo-box that would limit any twist under traffic that would otherwise be unacceptable. If plan bracing is needed, any connections to the flange should be bolted, to avoid the lower class fatigue details associated with the type of welded connections that would be needed.

10.5 Bearing details

10.5.1 Fabricated line rocker bearings

Line rocker bearings can be fabricated at modest cost and provide a long-life durable bearing. Typical arrangements are shown in Figure 10.23. The Figure shows a detail with positional freedom of the lower bearing plate – it permits rapid installation and allows adjustment in plan position before making the perimeter welds (note that the welds should be laid before the bridge is open to traffic, or there may be insufficient resistance to longitudinal or lateral loads).

The bearings are fastened to the main girder by means of bolts installed in threaded holes in the plate that is welded to the underside of the bottom flange. A similar detail can be used to fix the bearing to a bottom plate that is in turn

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site welded to a 'location plate', as illustrated in Figure 10.23. The use of a site weld allows freedom in positioning the bearing during installation.

For short spans (up to about 18 m) line rocker bearings can be used at both ends of the span, provided that there is sufficient clearance to the keep strips to permit thermal expansion/contraction.

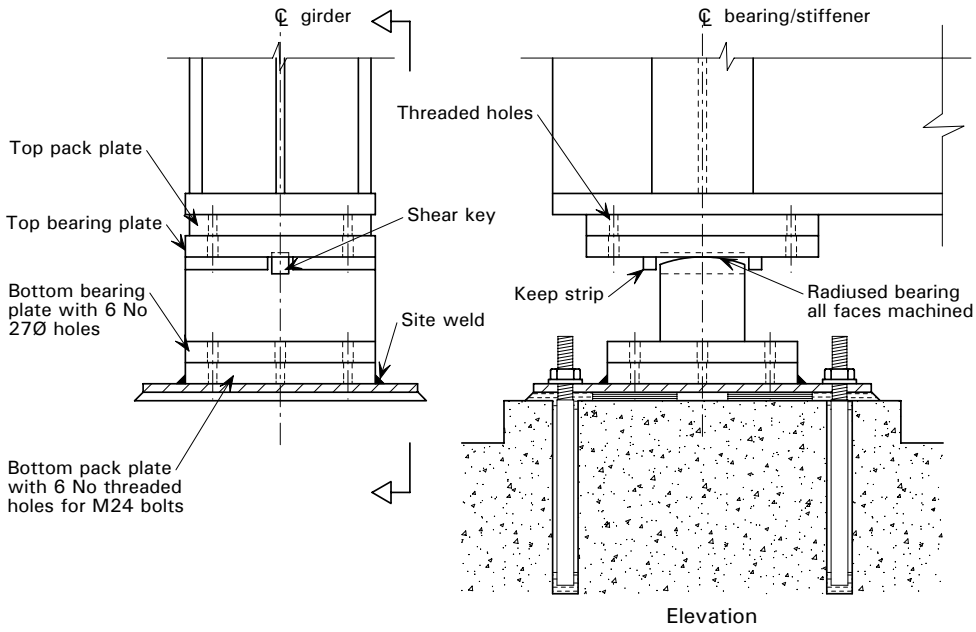

Figure 10.23 *Fabricated line rocker bearing*

10.5.2 Roller bearings

Where roller bearings are required, proprietary bearings should normally be specified.

10.5.3 Pot bearings

Pot bearings provide support without rotational restraint. They are commonly used on deck-type bridges and on longer half through bridges. Proprietary bearings should normally be specified and these will be fixed to top plates on the main girders and to bottom plates on the bearing shelf. For installation during a possession, the same freedom in plan position can be achieved by site welding the bottom plate, as for rocker bearings.

10.6 Deck end waterproofing details

The completion of the waterproofing system from the bridge to the abutment is usually a critical path activity. Installation of the components and accommodation of deviations from nominal geometry are key issues.

The use of pre-cast concrete ballast walls (on the abutment) are preferred by many railway contractors, particularly if the deck is being lifted into position as a single fully assembled element. The ballast walls can be fitted following the installation of the deck, allowing any tolerance issues between the deck and the sill beams to be taken up by the ballast walls. The ballast walls can be adjusted for line and level on mortar beds and fastened using locating bars. An important detail is then the continuity of the waterproofing membrane from the deck to the ballast wall. Various details have been developed for this and one

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example is shown simply in Figure 10.24. This shows a small bridging plate across the movement joint and an arrangement to continue the membrane over it and behind the ballast wall.

In choosing any arrangement at the abutment, it should be assumed that there is a strong probability that failure of the waterproofing across the movement joint will occur in service; leakage through any failure should be able to drain away from structural components, particularly any parts that are difficult to inspect and maintain. Bearings should be on plinths; drainage paths should be clear and not easily blocked by debris,

Whatever configuration is adopted, the details should be agreed at an early stage so that the steelwork can be suitably detailed and the erection scheme can be developed.

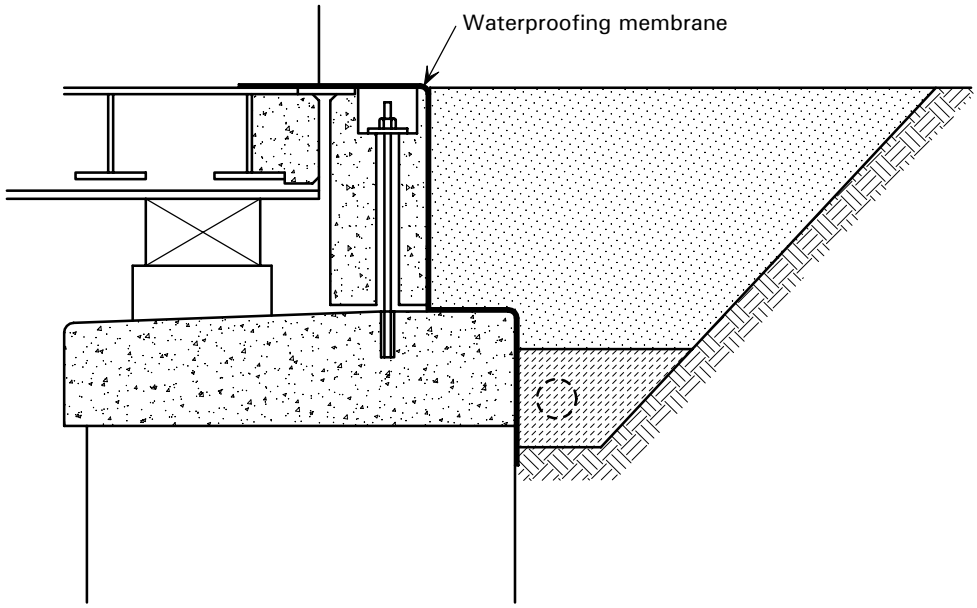

Figure 10.24 *Waterproofing at end of steel deck*

Water management, including waterproofing and drainage arrangements, is a key consideration at the ends of the deck. Account needs to be taken of likely flow rates and water disposal arrangements.

10.7 Footways

Where a cess walkway cannot be accommodated between the main girders of a half through bridge, it is likely that a footway will have to be provided on cantilever brackets outside the main girder. In many cases, services will also be required to be supported by the cantilever.

A typical arrangement is shown in Figure 10.25. Good line and level of the parapet rails is needed and this is best achieved by alignment during trial erection. To replicate the alignment during final erection, the detail shows a 'locating plate' that is positioned in contact with the web stiffener after the handrails have been aligned and then welded to the face of the bracket. Restoring this contact during final erection ensures good alignment.

HSFG bolts should be used for the brackets and for fastening the handrails, to ensure that slip does not take place in service; any slip would probably result in out-of-line handrails.

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The top surface of the main girders may be used as part of the walkway and, for safety, a non-slip surface should be provided. Details should be compatible with the protective treatment to the girders.

More substantial details are needed for supporting London Underground services.

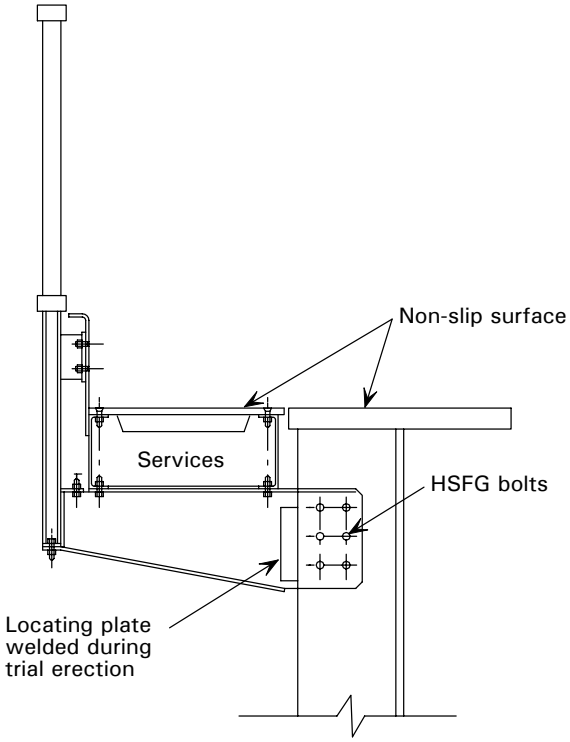

Figure 10.25 *Footway detail*

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APPENDIX A STEEL BRIDGE GROUP GUIDANCE NOTES

The following is a list of all the Guidance Notes in the SCI publication *Steel Bridge Group: Guidance notes on best practice in steel bridge construction*.

GN	Title
Section 1	Design – general
1.01	Glossary
1.02	Skew bridges
1.03	Bracing systems
1.04	Bridge articulation
1.06	Permanent formwork
1.07	Use of weather resistant steel
1.08	Box girder bridges
1.09	Comparison of bolted and welded splices
1.10	Half through bridges
Section 2	Design – detailing
2.01	Main girder make-up
2.02	Main girder connections
2.03	Bracing and cross-beam connections
2.04	Bearing stiffeners
2.05	Intermediate transverse web stiffeners
2.06	Connections made with HSFG bolts
2.07	Welds – how to specify
2.08	Attachment of bearings
Section 3	Materials and products
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3.02	Through thickness properties
3.03	Bridge bearings
3.04	Welding processes and consumables
3.05	Surface defects on steel materials
3.06	Internal defects in steel materials
3.07	Specifying steel material
Section 4	Contract documentation
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4.02	Weld procedure trials
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4.04	Alternative construction sequences
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5.01	Weld preparation
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5.03	Fabrication tolerances
5.04	Plate bending
5.05	Marking of steelwork
5.06	Flame cutting of structural steel
5.07	Straightening and flattening
5.08	Hole sizes and positions for HSFG bolts
5.09	The prefabrication meeting

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GN Title

Section 6 Inspection and testing
6.01 Weld quality and inspection
6.02 Surface inspection of welds
6.03 Sub-surface inspection of welds
6.04 Hydrogen/HAZ cracking and segregation cracking in welds
6.05 Weld defect acceptance levels

Section 7 Erection and in-situ construction work
7.01 Setting bearings
7.02 Temperature effects during construction
7.03 Verticality of webs at supports
7.04 Trial erection and temporary erection
7.05 Installation of HSFG bolts
7.06 Transport of steelwork by road
7.07 Site welding
7.08 Method statements

Section 8 Protective treatment
8.01 Preparing for effective corrosion protection
8.02 Protective treatment of bolts

Section 9 Other topics
9.01 Construction (Design and Management) Regulations

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APPENDIX B WORKED EXAMPLE

The worked example is of a twin-track bridge spanning 36 m. The bridge is square at its ends and the chosen form of construction is similar to an E type, except that the slab is wholly on top of the cross girders.

References in the margins are generally to clauses in BS 5400-3 although some references to Part 10 clauses are given in the format 10/9.1.2, as used in the main text of this publication.

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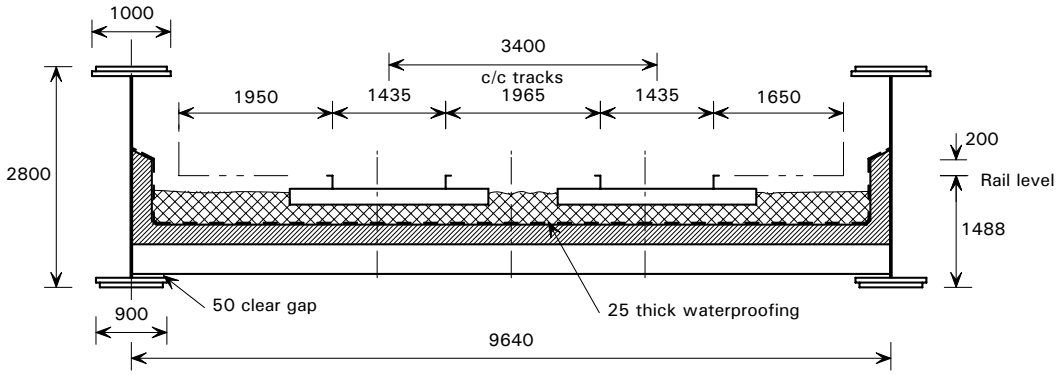
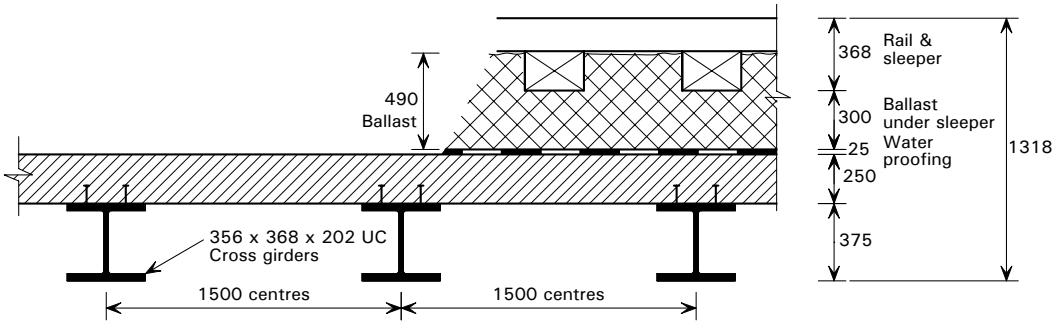
Job Title	Design guide for steel railway bridges			
Subject	Example: Design of girders for a half through plate girder bridge			
Client		Made by DCI	Date	Dec 2003
		Checked by CHP	Date	Feb 2004

General arrangement

Cross section

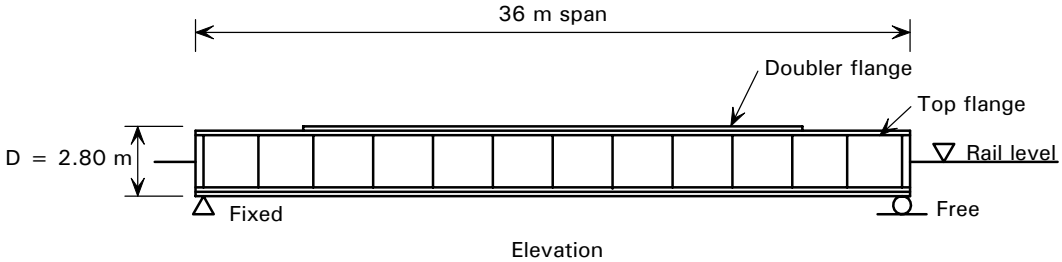


Longitudinal section on deck



Adjustment to the average ballast depth to allow for camber and drainage fall on deck is not shown in this example, for brevity.

Arrangement of doublers and web stiffeners



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Design standards

GC/RT5110
GC/RT5112
GC/RT 5203
GC/RC5510 (Note Appendix B, which modifies Parts 3, 4 and 5 of BS 5400)

BS 5400-3:2000
BS 5400-4:1990
BS 5400-5:1979 (including AMD 3998, May 1982)
BS 5400-10:1980 (including AMD 9352, March 1999) – use simplified procedure (see Clause 9.1.2).

BD 37/01 (Includes modified BS 5400-2)
DTP 'Yellow document' (BS 5400-3 plus BD 16/82)

Clause references are to BS 5400-3 unless noted otherwise

Design parameters

2 standard gauge tracks on straight alignment, track speed up to 100 mph.

A cess walkway is provided on one side of the track and a Continuous Position of Safety on the other side. The track is located with approximately 100 mm clearance between the outer edges of the walkway/CPOS and the inner edges of the top flange, to allow for possible future realignment of the track.

Heavy traffic, 27×10^6 tonnes/annum to be considered

Use grade S355 steel and grade C40 reinforced concrete

Combined response of bridge and track to variable loads (rail traffic loads and temperature) regarding track/bridge interaction does not need a specific check because rail expansion devices are to be provided at each end of the bridge. Thus longitudinal displacement of upper surface at end of deck due to temperature effects of the deck does not need to be checked.

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Loading

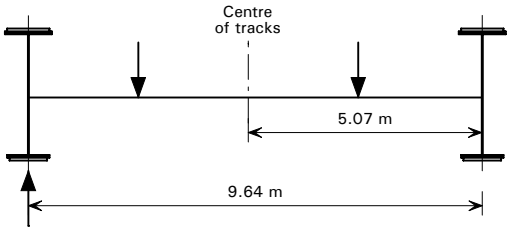
		Nominal kN/m	ULS Factor	ULS Load	SLS Factor	SLS Load
DEAD LOAD						
Main girder	0.267 m² × 77 kN/m³ plus 10%	22.61	1.10	24.87	1.00	22.61
Cross girders	2.04 kN/m × 4.80 m / 3.0 m crs	6.53	1.10	7.18	1.00	6.53
Slab	4.8 m × 0.25 m × 25 kN/m³	30.00	1.20	36.00	1.00	30.00
Haunch	0.96 m × 0.25 m × 25 kN/m³	6.00	1.20	7.20	1.00	6.00
Ballast	4.525 m × 0.49 m × 21 kN/m³	46.56	1.75	81.48	1.20	55.87
Track	2No × 2.0 kN/m × (5.07/9.64)	3.89	1.20	2.52	1.00	2.10
Waterproofing	(4.8+0.89 m) × 0.025 m × 24 kN/m³	3.41	1.75	5.97	1.00	3.41
Services	Say	0.80	1.20	0.96	1.00	0.80
				166.18		127.32

γ factors: BD 37/01 and GC/RC/5510

Ballast density GC/RC/5510

Track weight taken as 'extra over' ballast Allow 2kN/m for track with 3rd rail

Track position, relative to girders



		Nominal kN/m	ULS Factor	ULS Load	SLS Factor	SLS Load
LIVE LOAD						
EUDL	2 tracks × 4162 kN/36 m × (5.07/9.64)	121.68	1.40	170.35	1.1	133.85
Nosing (EUDL)	100 kN × 2/36m × 1.338/9.64	0.77	1.40	1.08	1.1	0.85
Walkway (far)	5 kN/m² × 0.7 m × 1.05/9.64	0.38	1.50	0.57	1.0	0.38
				172.00		135.08

EUDL: BD37 Table 27

		Nominal kN/m	ULS Factor	ULS Load
FATIGUE LOAD				
1 track	4162 kN × (6.77/9.64) / 36 m	81.23	1.00	81.23
2 tracks	2 × 4162 kN × (5.07/9.64) /36 m	121.68	1.00	121.68

Ratio 1 track / 2 tracks 81.23/121.68 0.67

BENDING MOMENTS

ULS Dead load	168.03 × 36²/8	26920 kNm
ULS Live load	172.00 × 36²/8	27860 kNm
		54770 kNm
SLS Dead load	127.32 × 36²/8	20630 kNm
SLS live load	135.08 × 36²/8	21880 kNm
		42500 kNm
Fatigue load	121.68 × 36²/8	19710 kNm

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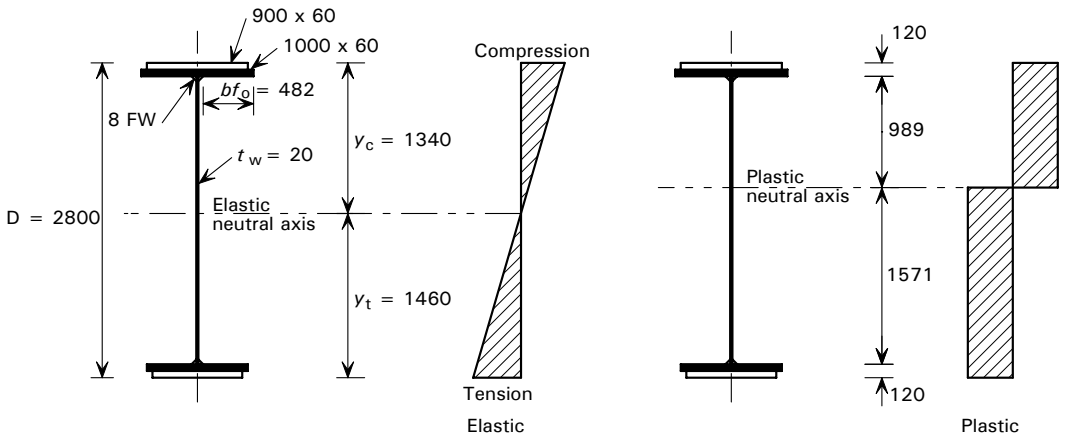
Main girder - section properties



Girder make-up

		yield strength
Doubler	900 × 60 mm	335 N/mm^2
Top flange	1000 × 60 mm	335 N/mm^2
Web	2560 × 20 mm	345 N/mm^2
Bottom flange	900 × 60 mm	335 N/mm^2
Bottom doubler	800 × 60 mm	335 N/mm^2

Elastic properties for vertical bending

I_{XX} = 0.4141 m^4 A = 0.2672 m^2
Z_{tfd} = 0.3091 m^3 y_{bfl} = 1.460 m
Z_{tf} = 0.3236 m^3 y_{tfl} = 1.340 m
Z_{bf} = 0.2957 m^3
Z_{bfd} = 0.2836 m^3

Plastic properties

M_{pe} 107600 kNm (based on yield strengths appropriate to element thicknesses)

$Z_{pe} = M_{pe}/335$ = 0.3212 m^3 y_{bfl} 1.691 m y_{tfl} 1.109 m

Shape limitations

b_{f0} = 482 mm > $7t_{fo}\sqrt{355/\sigma_y}$ = 420 × 1.029 = 432 mm Non-compact 9.3.7.3.1

Clear web depth = 2544 mm m = (1109 – 128)/2544 = 0.386 9.3.7.2

Limit for compact web: $\frac{34t_w}{m}\sqrt{\frac{355}{\sigma_{yw}}}$

= 680 × 1.014 / 0.386 = 1787 mm Non-compact

Effective sections

Web: $\frac{y_c}{t_w}\sqrt{\frac{\sigma_{yw}}{355}} = \frac{(1460-120)}{20}\sqrt{\frac{345}{355}} = 60 < 68$ Web fully effective 9.4.2.5.1

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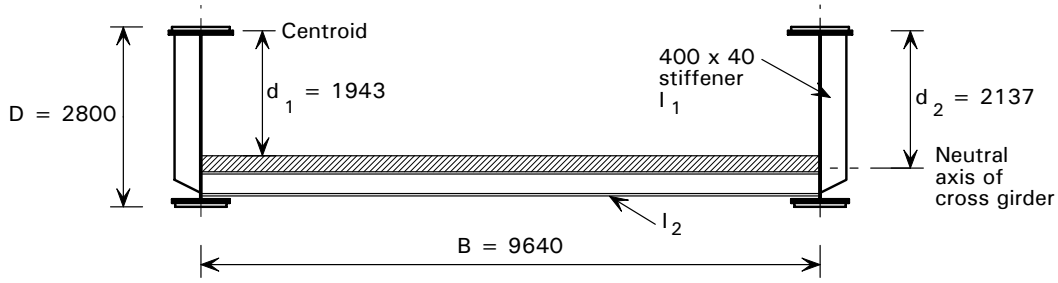
Intermediate U-frames – properties

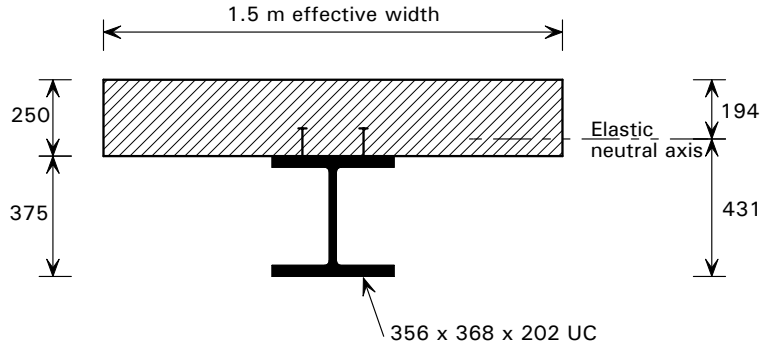
Stability of the compression flange is achieved by U-frame action. The configuration of the intermediate U-frames is:



Stiffness of crossbeams

Effective width of slab acting with crossbeam on each side is the lesser of: 9.6.4.1.3

$\ell_R/2 = 1.5/2 = 0.75$ m and
$B/8 = 9.64/8 = 1.205$ m
Effective width $= 2 \times 0.75 = 1.5$ m
Gross area of concrete within the effective width may be used



Concrete grade is C40, hence short term modulus $E_c = 31$ kN/mm^2 4/4.3.2.1

Modular ratio $= 205/31 = 6.61$

Second moment of area of composite section $I_2 = 0.003424$ m^4
$y_{steel} = 431$ mm
$y_{slab} = 194$ mm (NA in slab)

The neutral axis is just in the slab. Gross section properties can be used

The top of the slab is 250 + 375 + 120 + 50 = 795 mm above the main girder soffit

Take the centroid of the main girder top flange as 62 mm below the top of the doubler

Hence distance of centroid of top flange to top of slab:
$d_1 = 2800 - 62 - 795 = 1943$ mm

Distance from centroid of top flange to NA of cross girder and slab
$d_2 = 2800 - 62 - 795 + 194 = 2137$ mm

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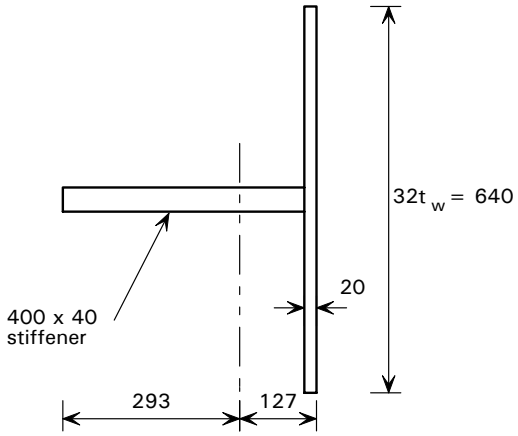
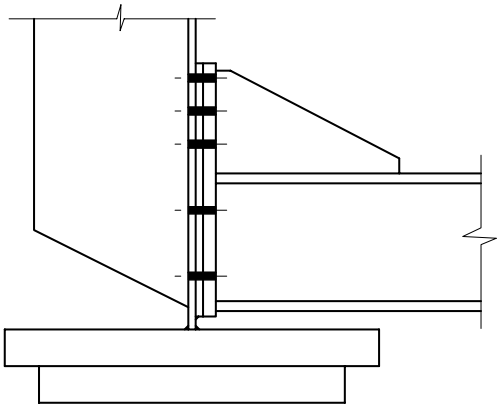
Stiffness of effective web stiffeners



$I_1 = 0.0005277\ m^4$
$A = 0.0288\ m^2$
$y_w = 127\ mm$
$y_s = 293\ mm$

Flexibility of cross girder/main girder connection



Consider the connection as the middle of the categories in clause 9.6.4.1.3 (i.e. comparable to those shown in Figure 42, Type (b). BS 5400-3

Hence $f = 0.2 \times 10^{-10}$ rad/Nmm $= 0.2 \times 10^{-4}$ rad/kNm

Lateral deflection of U-frame under unit loads is given by: 9.6.4.1.3

$$\delta_R = \frac{d_1^3}{3EI_1} + \frac{uBd_2^2}{EI_2} + fd_2^2$$

$$\delta_R = \frac{1.943^3}{3 \times 205 \times 10^6 \times 0.005277} + \frac{0.5 \times 9.64 \times 2.137^2}{205 \times 10^6 \times 0.004102} + 0.2 \times 10^{-4} \times 2.137^2$$

$$\delta_R = 0.0000226 + 0.0000314 + 0.0000913 = 0.0001453\ m/kN$$

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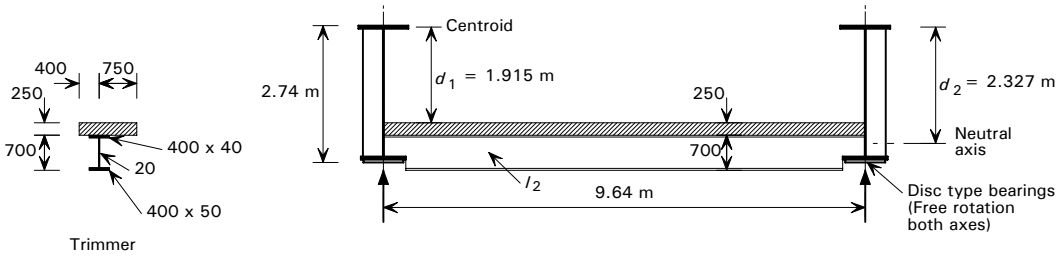
End U-frames – properties



Effective width of slab acting with trimmer = 0.75 +0.400 = 1.15 m

Second moment of area of composite section I_2 = 0.01022 m^4
Height of NA = 538 mm above bottom flange, 412 mm below top of slab

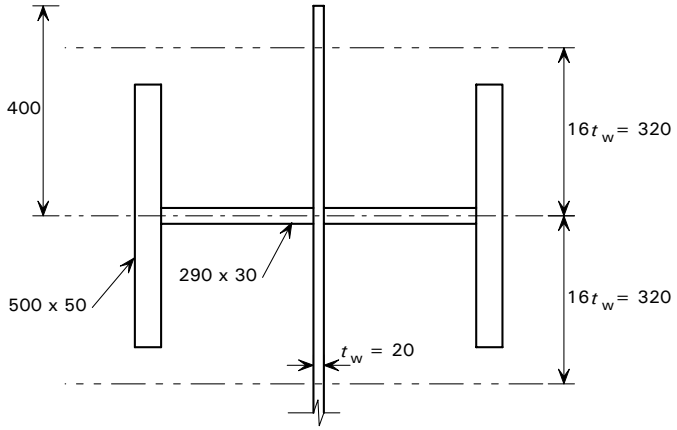
Top of slab is 795 mm above the soffit, as for the intermediate cross girders

Centroid of top flange is 90 mm below line of top of doubler (but no doubler at this section)

Hence d_1 = 2800 – 90 – 795 = 1915 mm

Hence d_2 = 2800 – 90 – 795 + 412 = 2327 mm

Stiffness of effective bearing stiffener



I_1 = 0.005833 m^4

Consider the connection of the trimmer to the main girder as the stiffest of the categories in clause 9.6.4.1.3 (i.e. comparable to those shown in Figure 42, Type (c). BS 5400-3

Hence f = 0.1 × 10^{-10} rad/Nmm = 0.1 × 10^{-4} rad/kNm

$\delta_{e,max}$ = δ_R for the end U-frame

$$\delta_R = \frac{d_1^3}{3EI_1} + \frac{uBd_2^2}{EI_2} + fd_2^2$$

$$\delta_{e,max} = \frac{1.915^3}{3 \times 205 \times 10^6 \times 0.005833} + \frac{0.5 \times 9.64 \times 2.327^2}{205 \times 10^6 \times 0.01022} + 0.1 \times 10^{-4} \times 2.327^2$$

= 0.0000020 + 0.0000125 + 0.0000541 = 0.0000686 m/kN

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Slenderness of main girders

$$\ell_e = k_2 k_3 k_5 \ell_1$$ 9.6.4.1.3

$$\ell_1 = \left(EI_c \ell_R \delta_R\right)^{0.25}$$ 9.6.4.1.1.2

I_c for a 1000 × 60 flange and 900 × 60 doubler $= 0.008645$ m^4

$$\ell_1 = \left(205 \times 10^6 \times 0.008645 \times 3.0 \times 0.0001453\right)^{0.25} = 5.272 \text{ m}$$

$$X = \frac{\ell_1^3}{\sqrt{2}\, EI_c \delta_{e,max}}$$

$$X = \frac{5.272^3}{\sqrt{2} \times 205 \times 10^6 \times 0.00865 \times 0.0000686} = 0.852$$

$$k_5 = 2.22 + \frac{0.69}{X + 0.5} = 2.730$$

$k_2 = 1.0$ (load on girder at bottom flange level)

$k_3 = 1.0$ (no rotation restraint in plan at supports)

$$\ell_e = 1.0 \times 1.0 \times 2.730 \times 5.272 = 14.39 \text{ m}$$

For slenderness of U-frames, λ_{LT} is given by: 9.7.2

$$\lambda_{LT} = \frac{\ell_e}{r_y}$$

The value of r_{yc} is that for the top flange plus one third of the depth of the web.
For that section:
$I_c = 0.008645$ m^4
$A = 0.131$
$r_{yc} = 0.257$ m

Hence

$$\lambda_{LT} = \frac{14.39}{0.257} = 56.0$$

Moment of resistance of main girder

Consider buckling mode:

$L/\ell_e = 36/14.39 = 2.50$ Hence there will be two half waves and $\ell_w = 18.0$ m

For input to Figure 11: BS 5400-3

$$\lambda_{LT}\sqrt{\left(\frac{\sigma_{yc}}{355}\right)\left(\frac{M_{ult}}{M_{pe}}\right)} = 56.0\sqrt{\left(\frac{335}{355}\right)\left(\frac{95010}{107600}\right)} = 52.6$$

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ℓ_e/ℓ_w = 14.39/18.00 = 0.80

From Figure 11b) (or Annex G) M_R/M_{ult} = 0.698

Hence M_R = 0.698 × 95010 = 66320 kNm

$M_D = M_R/\gamma_m\gamma_{f3}$ = 66320/(1.05 × 1.1) = 57420 kNm

This is slightly more than the applied moment and will probably be satisfactory but the combined effect with lateral bending still has to be checked. See sheet 25.

Check on dynamic performance

(*This check would normally follow immediately after calculation of loading and section properties, i.e. after Sheet 4 in this example, but is presented here for convenience*)

Dead load 127.5 kN/m (nominal DL + 100 mm extra ballast)

GC/RC5510 Appendix E

$$\text{Deflection} = \frac{5wL^4}{384EI} = \frac{5\times127.5\times36^4}{384\times205\times10^6\times0.414}\times1000 = 32.8\text{ mm}$$

This value is below the upper limit for a span of 36 m in Figure 1 of UIC 776-3R (*Corrected formula in GC/RC5510 gives a limit of 39 mm for a 36 m span*)

End rotation is given approximately by: 4 × 32.8 /36000 = 0.0036 rad
This value is below the limit in Table 1 of UIC 776-3R - OK

Live load (Nominal value, 2 tracks) 121.6 kN/m

$$\text{Deflection} = \frac{5wL^4}{384EI} = \frac{5\times121.6\times36^4}{384\times205\times10^6\times0.414}\times1000 = 31.3\text{ mm}$$

Span/deflection = 36000/31.3 = 11503

This ratio is better than the limit of 800 in Table 4 of UIC 776-3R for a single deck and for 'Speed Range 2'.

Vertical deflection

There is sufficient clearance that the vertical deflections do not infringe clearance requirements above the road below.

Details of deflection calculations are not presented here, for brevity, but the following were confirmed:

- The track twist with one (250 kN) axle 3 m onto the bridge on one track easily complies with twist limitations
- The limitation on uplift at the ends of the deck and rotation about a transverse axis at the end are also easily satisfied

UIC 776-3R

Lateral deflection is satisfactory by inspection

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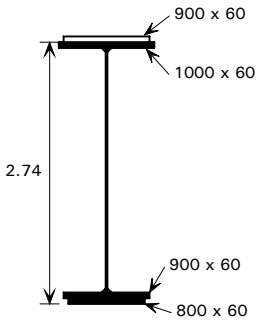
Curtailment of doubler plate

As well as the curtailment of the doubler plate within the span, the bottom flange is reduced in thickness at the ends (the flange will be fabricated by butt welding several lengths of plate and advantage can be taken to reduce the bottom flange size near the ends of the span).

Section properties for reduced girder section

I_{xx} = 0.2980 m^4 A = 0.2132 m^2
Z_{tf} = 0.1849 m^3 y_{bfl} = 1128 mm
Z_{bf} = 0.2789 m^3 y_{tfl} = 1612 mm
Z_{bfd} = 0.2641 m^3
M_{ult} = 61940 kNm

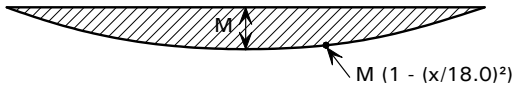


Using the slenderness derived for the full section, M_D will be approximately: $M_D = M_R/\gamma_m\gamma_{f3} = 0.698 \times 61940/(1.05 \times 1.1)$
$= 37430$ kNm

Allowing 5% to cover interaction with lateral bending, the section can be curtailed when $M = 0.95 \times 37430 = 35560$ kNm

Bending moment varies approximately as:

$$M_x = M_{centre} \times \left(1 - \left(\frac{2x}{L}\right)^2\right)$$



Where x is the distance from midspan

Hence the distance at which the doubler can be curtailed

$$x = 18.0 \times \sqrt{1 - \frac{35560}{54770}} = 10.66 \text{ m from midspan} \quad - \quad \text{Say 7.0 from each end}$$

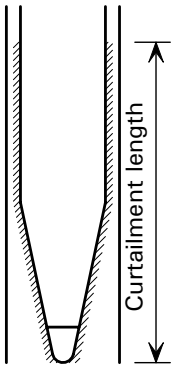
Length of curtailment

From above, the moment 7 m from the support = 34510 kNm

Average stress in the doubler plate:
= 34320 kNm /0.3164 m^3 = 108 N/mm^2

Load in doubler = 5832 kN

Assume 8 mm fillet welds along both sides. Around the nose of the doubler the weld size may be greater, depending on the taper in thickness of the doubler. That weld must be checked for fatigue (as Class W) – checks not presented in this example.



Tapers and radius as in Figure 10.5

Design strength of welds $= \dfrac{0.5(\sigma_y + 455)}{\sqrt{3}\,\gamma_m\gamma_{f3}} = \dfrac{0.5 \times (335 + 455))}{1.732 \times 1.05 \times 1.2} = 173 \text{ N/mm}^2$

14.6.3.11

Capacity of two 8 mm FW $= 2 \times \dfrac{8 \times 173}{\sqrt{2}} = 1957$ kN/m

Length required = 5832 / 1957 = 2.98 m Detail as 3.0 m long

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CALCULATION SHEET

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Check on curtailment of bottom flange

If the bottom flange doubler plate is curtailed within the span, the class G detail at the end of the doubler plate must be checked for adequacy in fatigue and the toughness of the bottom flange must also be adequate for the level of tensile stress at that location

For fatigue endurance, a similar location and curtailment detail to that on the top flange would be needed.

Properties for section without any doublers

I_{xx} = 0.2232 m^4 A = 0.1652 m^2
Z_{tf} = 0.1727 m^3 y_{bfl} = 1388 mm
Z_{bf} = 0.1609 m^3 y_{tfl} = 1292 mm
M_{ult} = 53900 kNm

Assume minimum effective bridge temperature U_e = –20°C

From Table 3c, maximum permitted thickness at –20°C is 60 mm for grades S355K2, S355N, S355M and 86 mm for grades S355NL and S355ML, for k = 1 — BS 5400-3

For the Class G detail, k_d = 0.5. Assume k_g = 1.0 and k_s = 1.0

The value of k_σ is given by Table 3b: — BS 5400-3
For $0.25\sigma_y < \sigma_{max} \leq 0.5\sigma_y$ $k_\sigma = 1.25$
For $\sigma_{max} \leq 0.25\sigma_y$ $k_\sigma = 1.5$
(σ_{max} tensile)

Hence the limiting thicknesses are:

For $0.25\sigma_y < \sigma_{max} \leq 0.5\sigma_y$
0.5 × 1.25 × 60 = 37.5 mm for grades K2, N, M
0.5 × 1.25 × 86 = 53.8 mm for grades NL, ML

For $\sigma_{max} \leq 0.25\sigma_y$
0.5 × 1.5 × 60 = 45 mm for grades K2, N, M
0.5 × 1.5 × 86 = 64.5 mm for grades NL, ML

To ensure that $\sigma_{max} \leq 0.25\sigma_y$ the total ULS moment must be limited to:
335 × 0.25 ×0.1609 = 13480 kNm

$$x = 18.0 \times \sqrt{1 - \frac{13480}{54770}} = 15.7 \text{ m from midspan}$$ Say 2.0 m from end of girder

Such a curtailment does not offer much saving in material and forces the use of the higher NL or ML grades. It will be more economic to carry the bottom doublers over the full span and to use grades K2, N or M.

For a bottom flange with a continuous doubler, the worst likely detail would be Class F, for which k_σ = 1.0 and at higher tensile stresses k = 1. The limiting thickness for toughness would then be 60 mm for grades S355K2, S355N, S355M (or slightly greater than 60 mm, following guidance in GN 3.08). — P185

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Shear resistance

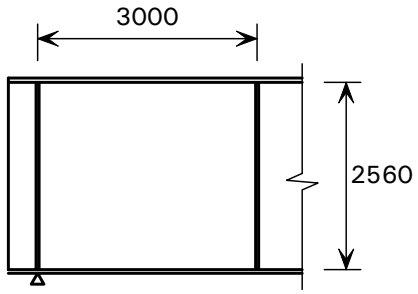
Loading at ULS

		Shear (kN)	
Dead load	166.18 × 36/2	2991	RU EUDL - BD37
Live load (RU EUL)	2 × (2094 × 1.4) × 5.07/9.64	3084	
Nosing	100 × 1.4 × 1.338/9.64	40	
		6115	

Clear Depth of web $d_{we} = 2560$ mm

Spacing of stiffeners $a = 3000$ mm

Flanges are parallel and straight, so use clause 9.9.2.2 — 9.9.2.1



$$\phi = \frac{3000}{2560} = 1.172$$

$$\lambda = \frac{d_{we}}{t_w}\sqrt{\frac{\sigma_{yw}}{355}} = \frac{2560}{20}\sqrt{\frac{345}{355}} = 126$$

For $m_{fw} = 0$, Figure 12 gives $\tau_\ell/\tau_u = 0.609$

Effective width of flange for m_{fw} (bottom flange is smaller if no doubler) — 9.9.2.2

b_{fe} = is the lesser of $10\,t_f\sqrt{\frac{355}{\sigma_{yf}}} = 10 \times 60\sqrt{\frac{355}{335}} = 618$ mm

and $b_f/2 = 450$ mm
Hence $b_{fe} = 450$ mm

$$m_{fw} = \frac{\sigma_{yf} b_{fe} t_f^2}{2\sigma_{yw} d_{we}^2 t_w} = \frac{335 \times 450 \times 60^2}{2 \times 345 \times 2560^2 \times 20} = 0.00600$$

For $m_{fw} = 0.005$, Figure 13 gives $\tau_\ell/\tau_u = 0.695$ and for $m_{fw} = 0.0105$, Figure 14 gives $\tau_\ell/\tau_u = 0.790$

So for $m_{fw} = 0.00600$ $\tau_\ell/\tau_u = 0.714$

$\tau_u = 345/\sqrt{3} = 199$ N/mm^2

$\tau_\ell = 0.714 \times 199 = 142$ N/mm^2

$$V_D = \left[\frac{t_w(d_w - h_h)}{\gamma_m \gamma_{f3}}\right]\tau_\ell = \left[\frac{20 \times (2560 - 0)}{1.05 \times 1.1}\right] \times 142 \times 10^{-3} = 6294 \text{ kN}$$ — 9.9.2.2

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V_D > 6115 kN so this is just satisfactory. However, the value of shear stress at which tension field action starts, τ_o is only 78 N/mm^2 and the difference between that and the stress at ULS (120 N/mm^2) gives rise to large tension field forces and to large forces on the intermediate stiffener and on the endpost. To reduce those forces, it is better to increase the web thickness to 22 mm (additional intermediate stiffeners could be added in the two panels at each end but this creates extra U-frames and a slightly thicker web locally is more economic).

For a 22 mm web

$$\lambda = \frac{d_{we}}{t_w}\sqrt{\frac{\sigma_{yw}}{355}} = \frac{2560}{22}\sqrt{\frac{345}{355}} = 115$$

$m_{fw} = 0.00546$

For $m_{fw} = 0$, Figure 12 gives $\tau_\ell/\tau_u = 0.680$
For $m_{fw} = 0.005$, Figure 13 gives $\tau_\ell/\tau_u = 0.795$ and
For $m_{fw} = 0.010$, Figure 14 gives $\tau_\ell/\tau_u = 0.831$

So for $m_{fw} = 0.00546$ $\tau_\ell/\tau_u = 0.798$ and $\tau_\ell = 0.798 \times 199 = 159$ N/mm^2

$$V_D = \left[\frac{22\times(2560-0)}{1.05\times1.1}\right]\times159\times10^{-3} = 7753 \text{ kN}$$

At the support, 9.14.3.4

$$\tau_o = 3.6E\left\{1+\left(\frac{b}{a}\right)^2\right\}\left(\frac{t_w}{b}\right)^2 = 3.6\times205000\left\{1+\left(\frac{2560}{3000}\right)^2\right\}\left(\frac{22}{2650}\right)^2 = 94 \text{ N/mm}^2$$

$\tau = 6115/(2560 \times 22) = 109$ N/mm^2

The end support must be designed to resist an additional moment M_y given by:

$$M_y = \frac{8(\tau-\tau_o)t_w\ell_s^2}{\theta_d} =$$

θ_d = arctan(2560/3000) = 40.5 degrees

$$M_y = \frac{8\times(109-94)\times22\times2560^2}{40.5}\times10^{-6} = 427 \text{ kNm}$$

This moment is easily resisted by using an endplate to the girder, of a similar size to the bearing stiffeners and at a distance of 400 mm from the bearing stiffener.

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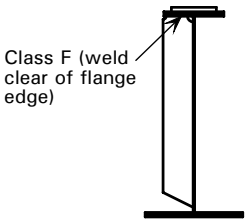
Fatigue – main girder

At midspan

The top flange is 'Class F' due to the transverse stiffener to flange weld on the underside and due to the doubler to flange fillet weld on the upper side. The bottom flange is Class D, due to the web/flange weld, but the web is class G at the patch plates for the cross girders and is slightly further from the NA than the top flange.



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Moment due to unfactored live load at mid span

$M = 121.7 \times 36^2/8 = 19720$ kNm — Sheet 3

Elastic modulus at top of bottom flange $= 4.141 \times 10^{11} / 1340 = 3.090 \times 10^8$ mm^3 — Sheet 4

Stress range $\sigma_R = \dfrac{19720 \times 10^6}{3.090 \times 10^8} = 64$ N/mm^2 $= \sigma_{Rmax}$

Limiting stress range $\sigma_T = k_1 \times k_2 \times k_3 \times k_4 \times k_5 \times \sigma_o$ — 10/9.2.2.1

$k_1 = 1.0$ (design life 120 years)

$k_2 = 1.0$ (single cycle of loading)

$k_3 = 2.19$ (length of influence line > 28 m, heavy traffic)

$k_4 = 1.0$ (27 million tonnes/year)

$k_5 = 1.27$ (Since $P_1/(P_1 + P_2) = 6.773/(6.773 + 3.372) = 0.668$) — 10/Table 6

For Class G, $\sigma_o = 29$ N/mm^2 — 10/Table 8

$\sigma_T = 1.0 \times 1.0 \times 2.19 \times 1.0 \times 1.27 \times 29 = 81$ N/mm^2 > 64 N/mm^2 OK

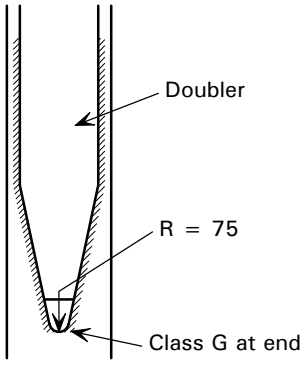
At the end of the doubler

The top flange plate is Class G

The end of the doubler is 4.0 m from the end of the span, i.e. 14.0 m from midspan

The moment due to unfactored live load can be taken as:

Moment at midspan × [1 – (14.0/18.0)^2]
= 19720 × 0.395 = 7791 kNm



Elastic modulus = 1.849×10^{11} mm^3 — Sheet 10

Stress range $\sigma_R = \dfrac{7791 \times 10^6}{1.849 \times 10^8} = 42$ N/mm^2 $= \sigma_{Rmax}$

For Class G, $\sigma_o = 29$ N/mm^2

$\sigma_T = 1.0 \times 1.0 \times 2.19 \times 1.0 \times 1.27 \times 29 = 80$ N/mm^2 > 42 N/mm^2 OK

Note that, for the class G detail in compression, $k_d = 0.5$ and $k_\sigma = 2$, which gives a limiting thickness of 60 mm for grades S355 K2, N and M, which is adequate. — 6.5.3, 6.5.4

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Welds between web and flange at the support

ULS loading

Shear at support = 6115 kN (Sheet 12)

Shear flow in web is given by:

$$q = \frac{VAy}{I}$$

Where A is the area and y is the distance to its centroid, of the part of the section further from the neutral axis than the position being considered.

Elastic properties of reduced main girder section (with bottom doubler & 20 mm web) Sheet 10

I_{XX}	= 0.2980 m^4	A	= 0.1654 m^2
Z_{tf}	= 0.1849 m^3	y_{bfl}	= 1.128 m
Z_{bf}	= 0.2789 m^3	y_{tfl}	= 1.612 m
Z_{bfd}	= 0.2641 m^3		

The bottom flange is larger than the top flange, so consider the shear flow at the web/flange junction of the reduced girder section (i.e. without doubler).

Area of bottom flange	= 0.054 m^2
Area of bottom flange doubler	= 0.048 m^2
Distance to centroid of bottom flange	= 1.038 m
Distance to centroid of bottom flange doubler	= 1.098 m

$$q = \frac{6115 \times (0.054 \times 1.038 + 0.048 \times 1.098)}{0.298} = 2316\,\text{kN/m}$$

Shear capacity of two 8 mm fillet welds = 1957 kN/m (see calculations for doubler plate on Sheet 10).

Shear capacity of two 10 mm fillet welds = 2447 kN/m 10 mm welds satisfactory

Note that in earlier calculations, the dimensions of the welds were assumed to be 8 mm leg length. The larger size has negligible effect on those calculations and in practice, smaller deeper penetration welds might be used, in which case the 8 mm dimension would still be appropriate.

Fatigue loading

Live load shear = 2 × 2094 × 5.07/9.64 = 2204 kN

Range of shear flow: 2329 × (2204/6149) = 835 kN/m

Stress range = 835 / (2 × 10 × 0.7071) = 59 N/mm^2

For Class W, σ_o = 25 N/mm^2

k factors are as for other main girder checks (Sheet 14)

σ_T = 1.0 × 1.0 × 2.19 × 1.0 × 1.27 × 25 = 70 N/mm^2 > 59 N/mm^2 OK

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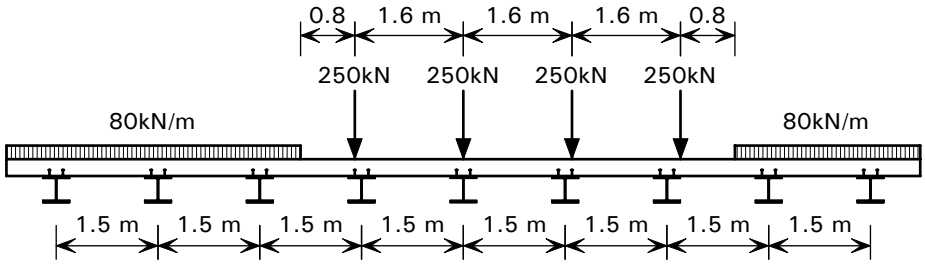
Cross girder - loading

Live loading

Consider the proportion of live loading carried by a single cross girder. Assume worst effect is with one of the concentrated loads over a cross girder



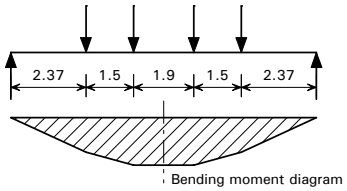
Loading on the central cross girder may be estimated initially by simple 'rule of thumb' distribution from consideration of 'statics' reactions but will need to be verified by suitable analysis. Here, assume that the loading is simply the load due to one of the concentrated edge loads (i.e. 250 kN/track)
(A grillage model showed this assumption to be a slight overestimate in this case)

For dynamic factor, take length of influence line for deflection as twice the main girder spacing — BS 5400-2 Table 17

$$\text{Dynamic factor} = 0.73 + \frac{2.16}{\sqrt{19.28} - 0.2} = 1.245$$

Loading on a single cross girder, from one track: $250 \times 1.245 = 311$ kN

For 'worst case' loading on a cross girder, assume tracks centrally located and assume that the cross girder is simply supported. Bending moment in cross girder, due to loads applied along the lines of the rails (1.5 m spacing of wheel loads, inner rails 1.9 m spacing) is given by:



$M = 311 \times (2.37 + 3.87) \times \gamma_{fL} = 970 \times 1.4 = 1358$ kNm

Dead loads

		Nominal	ULS		SLS	
Total loads		kN	Factor	Load (kN)	Factor	Load (kN)
Cross girders	9.6 m × 2.04 kN/m	19.58	1.10	21.50	1.0	19.58
Slab	1.5 × 9.6 m × 0.25 m × 25 kN/m^3	90.00	1.2	108.00	1.0	90.00
Haunch	2No 1.5 × 0.96 m × 0.25 m × 25 kN/m^3	18.00	1.20	20.70	1.0	18.00
Ballast	1.5 × 9.05 × 0.49 m × 21 kN/m^3	139.69	1.75	244.50	1.2	167.63
Track	2No × 3 m × 2.0 kN/m	6.00	1.2	7.20	1.0	6.000
Waterproofing	1.5 × (9.6+0.89) × 0.025 × 24 kN/m^3	9.75	1.75	17.10	1.2	11.70
				419.90		312.91

Calculate ULS moment due to this total load as a UDL (conservative)
$M = 419.9 \times 9.64 / 8 = 506$ kNm

Total ULS moment $= 1358 + 506 = 1864$ kNm

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Section properties of cross girder

Beam section is compact, by inspection

For concrete strength 40 N/mm^2 and steel yield strength 345 N/mm^2, the equivalent steel area of the slab in a plastic section is:

$1500 \times 250 \times (0.4 \times 40 \times \gamma_m / 345) = 18260$ mm^2 (γ_m for steel = 1.05) 5*/6.2.2

Area of UC section = 25700 mm^2

From a balance of tensile and compressive areas, the plastic neutral axis is 10 mm into the top flange of the UC. M_{pe} is then given by:

$M_{pe} = 345 \times (18260 \times 135 + 25700 \times 177 - 2 \times 7440 \times 5) \times 10^{-6} = 2407$ kNm

$M_D = 2407/1.1 \times 1.05 = 2084$ kNm > 1870 kNm OK

Elastic section properties for SLS and fatigue checks

Effective breadth ratio ψ depends on b/L

$b/L = 0.75 / 9.64 = 0.077$

For fatigue loading, consider the loading under the inner rail of one track (position of maximum moment from one track). Determine effective breadth for quarter span regions (slightly conservative).

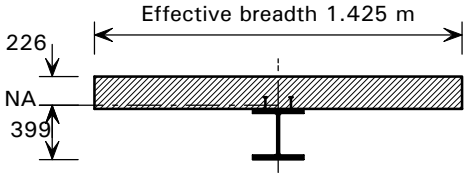
From Table 4, with $b/L = 0.077$, $\psi = 0.95$

Effective width of slab = $2 \times 750 \times 0.95 = 1425$ mm

Concrete grade is C40, hence short term modulus $E_c = 31$ kN/mm^2

Modular ratio = 205/31 = 6.61

Section properties of composite section
$I_2 = 0.002634$ m^4
$y_{steel} = 399$ mm
$y_{slab} = 226$ mm (NA just in slab)
$Z_{bf} = 6.602 \times 10^6$ mm^3
$Z_{tf} = 108.8 \times 10^6$ mm^3



For SLS checks (required because the section is considered compact at ULS) a slightly greater effective width can be used for midspan regions, where the applied moment is greatest, but there is little difference in section properties

Check at SLS

Average load factor is at least 1.27 times greater at ULS than at SLS, so an upper bound for SLS design moment would be $1864/(1.27 \times 1.1 \times 1.05) = 1268$ kNm

Stress in bottom flange = 1268 / 6.602 = 192 N/mm^2 Satisfactory

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Fatigue – cross girder

Maximum moment due to loading from one track (take track as centrally located)

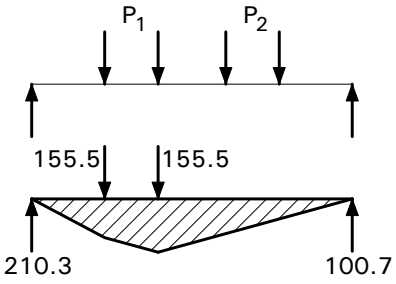
Loading from one track, including dynamic factor = 311 kN — Sheet 16

Reaction at RH main girder, due to load on LH track = 311 × 3.12 / 9.64 = 100.7 kN

Moment at inner rail = 100.7 × 5.77 = 581 kNm — Sheet 16

Total unfactored moment due to two tracks
M = 311 × 3.12 = 970 kNm

Hence $P_1 / (P_1 + P_2)$ = 581 /970 = 0.60



BMD for one track loaded

Stress range at top flange level

σ = 970 10^6 / 108.8 × 10^6 = 9 N/mm^2

Stress range at bottom flange level

σ = 970 × 10^6 / 6.602 × 10^6 = 147 N/mm^2

Limiting stress range

$\sigma_T = k_1 \times k_2 \times k_3 \times k_4 \times k_5 \times \sigma_o$

k_1 = 1.0 (design life 120 years) — 10/9.2.2.1d

k_2 = 1.0 (single cycle of loading)

k_3 = 1.92 (length of influence line = 19.28 m, heavy traffic) — 10/Table 4

k_4 = 1.0 (27 million tonnes/year) — 10/Table 5

k_5 = 1.27 (Since $P_1/(P_1 + P_2)$ = 0.60) — 10/Table 6

Top flange is Class F (shear studs are welded to flange), σ_o = 40 N/mm^2 — 10/Table 8

σ_T = 1.0 × 1.0 × 1.92 × 1.0 × 1.27 × 40 = 98 N/mm^2 > 9 N/mm^2 OK

Bottom flange is Class C (rolled section, as rolled condition, no welds)
σ_o = 40 N/mm^2

σ_T = 1.0 × 1.0 × 1.92 × 1.0 × 1.27 × 78 = 190 N/mm^2 > 147 N/mm^2 OK

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CALCULATION SHEET

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Job Title	Design guide for steel railway bridges			
Subject	Example: Design of girders for a half through plate girder bridge			
Client		Made by DCI	Date	Dec 2003
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Connection of cross girders to main girders

The magnitude of the design moment at the connection depends on the values of the restraint forces F_R and F_c — 9.12.2, 9.12.3.3

The force F_R is given by:

$$F_R = \left(\frac{\sigma_{fc}}{\sigma_{ci}-\sigma_{fc}}\right)\frac{\ell_w}{667\delta_R} \text{ but not greater than } \left(\frac{\sigma_{fc}}{\sigma_{ci}-\sigma_{fc}}\right)\frac{EI_c}{16.7\ell_R^2}$$

Consider the stress in the flange as that due to ULS loading (Sheet 3)

$\sigma_{fc} = 54770 \times 10^6 / 3.091 \times 10^6 = 177$ N/mm^2

$$\sigma_{ci} = \frac{\pi^2 ES}{\lambda_{LT}^2}$$

S = 0.3212/0.3091 = 1.039

λ_{LT} = 56 (Sheet 8)

$$\sigma_{ci} = \frac{\pi^2 \times 205000 \times 1.039}{56^2} = 670 \text{ N/mm}^2$$

δ_R = 0.0001453 mm/N (Sheet 6)

ℓ_w = 18.0 m (Sheet 8)

I_c = 0.008645 m^4 (Sheet 8)

$$F_R = \left(\frac{177}{670-177}\right)\frac{18.0}{667\times 0.0001453} = 66.7 \text{ kN}$$

But not more than

$$F_R = \left(\frac{177}{670-177}\right)\frac{205000\times 8.645\times 10^9}{16.7\times 3000^2} = 4233 \text{ kN}$$

Hence F_R = 66.7 kN

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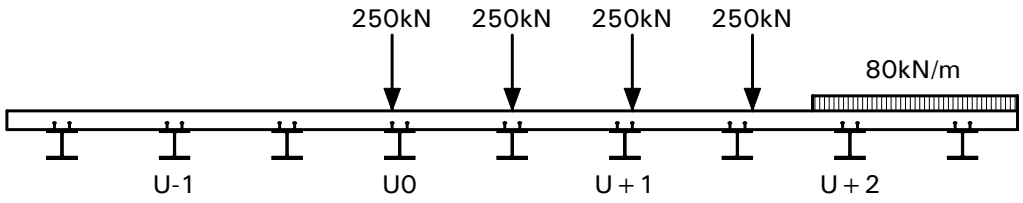
The force F_c depends on the rotation θ of the end of the cross girder, relative to those either side of it, when the cross girders are all simply supported at their ends.

$$F_c = \frac{\theta\, d_2}{1.5\delta_R + \frac{\ell_R^2}{12EI_c}}$$

Consider the cross girders at midspan, under a loading arrangement as below:



The UDL is omitted on one side as it gives a worse differential loading between adjacent U-frames. Simple 'static distribution' with 30% redistribution to adjacent cross girders gives loading at U-1, U0, U+1 and U+2 to be 0, 209 kN, 234 kN and 150 kN. (A grillage analysis would given similar values but with a little more distribution to frame U-1).

The average of the values at U-1 and U+1 is 117 kN, so the 'differential loading' on U0 is 209- 117 = 92 kN.

Using the above estimate of loading that causes relative displacements, the end rotations may be calculated simply by equating them to the area of the M/EI diagram between the middle and the end of the cross girder. Use the properties of the effective section for U-frame action (Sheet 5).

In this case, the diagram is polygonally shaped and the area can be taken approximately as 2/3 height times length.

The live load midspan bending moment on the most heavily loaded cross girder is derived on Sheet 16 as 1358 kNm so the applicable moment for relative loading is:
92/250 × 1358 = 500 kNm

θ = 2/3 ×500 × 4.82 / 205 × 10^6 × 0.002634 = 0.00297 radians

d_2 = 2137 mm (Sheet 5) and depth to mid-depth of UC = 2381 mm

$$F_c = \frac{0.00297 \times 2137}{1.5 \times 0.0001453 + \frac{3000^3}{12 \times 205000 \times 8.645 \times 10^9}} \times 10^{-3} = 29.0 \text{ kN (ULS value)}$$

The moment at the endplate connection is the product of the force (F_R + F_c) and the lever arm from the centroid of the top flange to the mid-depth of the UC.

Hence moment on connection:
M = (66.7 + 29.0) × 2.381 = 228 kNm at ULS
And M = (66.7 × 42000/55080 + 29.0 × 1.1/1.4) × 2.381 = 178 kNm at SLS

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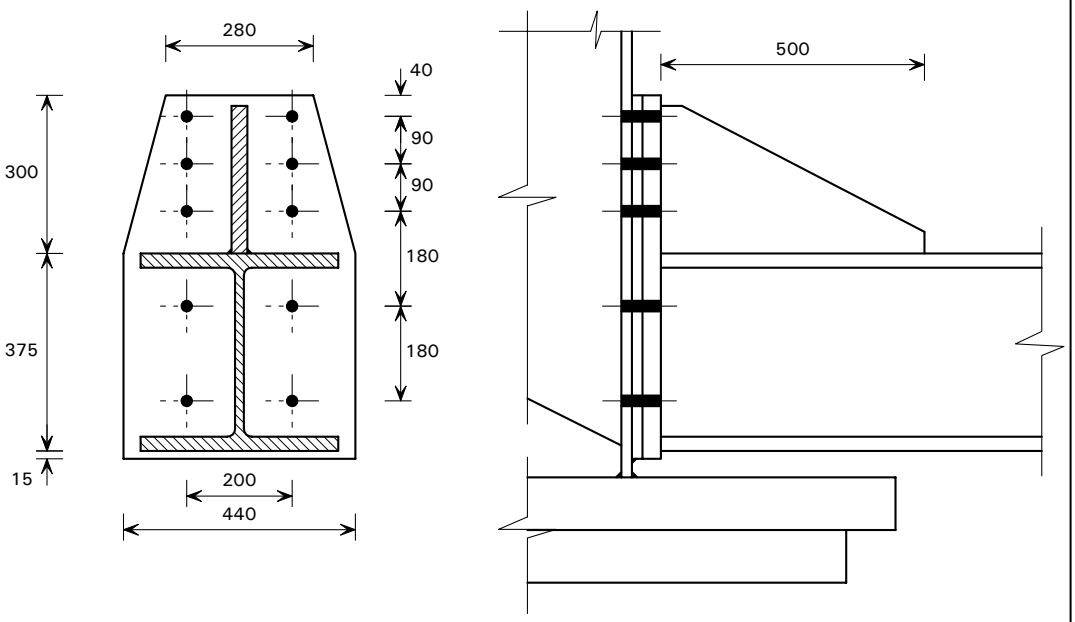
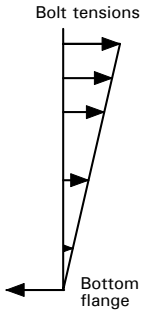
Bolted connection



Consider the hogging moment at the connection of frame U0

Check tension in top row of bolts (SLS)



'Inertia' of bolts about centre of bottom flange (for hogging bending)

$I_{bolts} = 2 \times (0.081^2 + 0.261^2 + 0.441^2 + 0.531^2 + 0.621^2)$
$= 1.874\ m^2$

Moment from Sheet 20

SLS load at top row of bolts $= 178 \times 0.621/1.874 = 59$ kN
Add 10% for prying Load = 65 kN

14.3.6

Preload 207 kN (M24 general grade) OK

Check shear at interface (SLS)

Dead load shear at end of cross girder = 156 kN (Sheet 16)

Dynamic factor for shear $= 0.82 + \dfrac{1.44}{\sqrt{19.28} - 0.2} = 1.164$

Live load shear $2 \times 209 \times 1.164 \times 5.07 / 9.64 \times 1.1 = 282$ kN

Total shear $156 + 282 = 438$ kN

N = 10 bolts Shear / bolt = 43.8 kN

$$P_D = \frac{k_h F_v \mu N}{\gamma_m \gamma_{f3}}$$

14.5.4.2

$k_h = 1.0$ and $\mu = 0.5$ For resistance of a single bolt, $N = 1$

Prestress load $F_v = 207 - 65 = 142$ kN

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$$P_D = \frac{1.0 \times 142 \times 0.5 \times 1}{1.2 \times 1.0} = 59.2 \text{ kN} > 43.8 \text{ kN} \quad \text{OK}$$

Shear at ULS is OK by inspection, since the prestress load is greater and the applied shear is less than (1.4/1.1) times the SLS shear but combined vertical and longitudinal shear needs to be checked.

Consider the sagging moment at the connection of frame U-1

Check tension in bottom row of bolts (SLS)

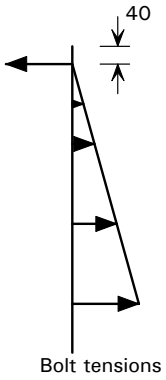
'Inertia' of bolts about top row of bolts (assuming that this is the effective line of action of the compressive component of the moment)

$I_{bolts} = 2 \times (0.090^2 + 0.180^2 + 0.360^2 + 0.540^2) = 0.924 \text{ m}^2$

F_c and F_R forces are as for frame U0 but in outward direction

SLS load at bottom row of bolts = $178 \times 0.540/0.924 = 104$ kN
Add 10% for prying Load = 114 kN

Preload 207 kN (M24 general grade HSFG) OK



Check shear at interface (SLS)

Dead load shear at end of cross girder = 156 kN (Sheet 16)

Live load shear = 0 (no load on this crossframe in the sagging moment condition)

Shear / bolt = 156/10 = 15.6 kN

Prestress load F_v = 207 – 114 = 93 kN

$$P_D = \frac{1.0 \times 93 \times 0.5 \times 10}{1.2 \times 1.0} = 38.8 \text{ kN} > 15.6 \text{ kN} \quad \text{OK}$$

Check bending of endplate at ULS

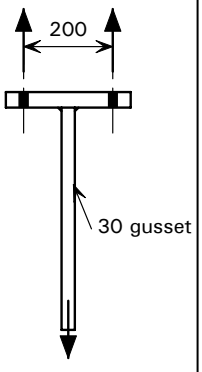
Moment on endplate M = 228 kNm (Sheet 20)

ULS load at top row of bolts = $228 \times 0.621/1.874 = 76$ kN

Moment at face of gusset (assuming 'cantilever bending' of rigid endplate and allow 10% for prying action)
$M = 76 \times 0.085 \times 1.10 = 7.11$ kNm

Effective width of plate = 40 + 90/2 = 85 mm
(top of endplate to midway between bolts)

Assuming a 35 mm thick endplate



Plastic moment capacity of 85 mm width $Z_{pe} = 35^2 \times 85/4 = 26000 \text{ mm}^3$

Bending resistance $M_D = 26000 \times 345 / 1.05 \times 1.1 = 7.78$ kNm > 7.11 kNm OK

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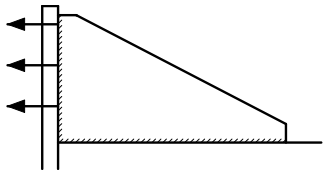
Requirement for patch plate on main girder

There will be a similar bending in the main girder web plate, so a similar total thickness will be required. Web thickness = 20 mm, so weld on a 20 mm patch plate

Check welds to gusset plate, under tensile forces (ULS)



Bolt forces = 76 kN, 65 kN and 54 kN in the top three rows (no extra 10%)

Total force = $195 \times 2 = 390$ kN

Load on weld to end plate = 76 kN / 85 mm
= 894 kN/m per side

Capacity of 8 mm FW = 979 kN/m (Sheet 10) OK

Shear on attachment to UC = 390 kN / (2 sides × 500 mm) = 390 kN/m

Bending on horizontal interface

$M = 2 \times (76 \times 0.260 + 65 \times 0.170 + 54 \times 0.080) = 70.4$ kNm

Z of 500 mm long welds = $2 \times 0.50^2 / 6 = 0.0833$ m^2

Bending load on weld = 70.4 / 0.0833 = 845 kN/m

Resultant load = $\sqrt{390^2 + 845^2}$ = 931 kN/m

< Capacity of 8 mm FW (979 kN/m, Sheet 10) OK

By inspection, a 8 mm FW between the web stiffener and the web is likely to be adequate to transmit the loading from the bolts, via the web, into the stiffener. Detailed checks should be made on that connection (calculations not included here).

Fatigue loading on welds

The fatigue loading on the welds can be taken very conservatively as 1/1.4 times the above ULS loading (the loads that give rise the calculated forces in the welds include a significant contribution from the dead loading on the main girder), that is 931/1.4 = 665 kN/mm

Stress in two 8 mm fillet welds = $665/(2 \times 5.66) = 59$ N/mm^2

The limiting stress range is derived as on Sheet 18, using the same k factors and a value of $\sigma_0 = 25$ N/mm^2 (for a class W detail)

$\sigma_T = 1.0 \times 1.0 \times 1.92 \times 1.0 \times 1.27 \times 25 = 61$ N/mm^2 > 59 N/mm^2 OK

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Cross girder connection not part of U-frame

Vertical shear

Dead load shear (SLS) = 156 kN (as for U-frame cross girders)

Live load shear (for worst loading) = 311 × 1.1 = 342 kN (Sheet 16)

Total load = 156 + 342 = 498 kN

Slip resistance per bolt (M24 HSFG)

$$P_D = \frac{1.0 \times 207 \times 0.5 \times 10}{1.2 \times 1.0} = 86.3 \text{ kN}$$

Capacity of 6 bolts = 518 kN OK

A 25 mm (minimum) endplate will be used

The weld between UC web and endplate will need to be checked for adequacy in fatigue. A 10 mm FW will probably be OK (calculations not presented here).

Longitudinal shear

Shear lag factor at SLS is given by b/L ratio

b = 4.8 m hence b/L = 4.82/36.0 = 0.15

ψ = 0.46 (Table 4) — Table 4 of Part 3

For calculation of shear flow at support, assume slab is uncracked at the ends

Section properties for long-term and short-term composite sections give the following:

$\frac{Ay}{I} = 5.23 \times 10^{-5}$ mm^{-1} (long term) and $\frac{Ay}{I} = 8.63 \times 10^{-5}$ mm^{-1} (short term)

(*Section properties not presented here*)

Dead load shear = 127.32 × 36/2 = 2292 kN

Shear flow = 2292 × 5.23 × 10^{-5} = 120 kN/m

Live load shear = 2094 × 2 × 5.07/9.64 = 1101 kN — RU EUDL BD37

Shear flow = 1101 × 8.63 × 10^{-5} = 95 kN/m

Total shear flow = 120 + 95 = 215 kN/m

Shear on first non U-frame cross girder will be the most onerous

Longitudinal shear = 215 × 1.5 = 323 kN

This load is too great to be carried in friction at the same time as the vertical shear. A shear plate detail will be required (see Figure 10.12). The shear plate will transfer vertical shear in bearing and longitudinal shear in friction – 6 bolts should be provided.

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Lateral bending of main girder top flange

Now that the relative rotation of the most heavily loaded cross girder has been obtained (θ = 0.00297 radians), the lateral bending of the top flange can be evaluated, using Appendix E of Part 3. 9.12.3.3(b)

$$M_y = \frac{5EI_c\theta d_2}{L\ell_e(1-\sigma_{fc}/\sigma'_{ci})}\left\{1+\frac{(L/\ell_e)-1.25}{2.8+3.5(\sigma_{fc}/\sigma'_{ci})^2}\right\}$$

From Sheet 8:
ℓ_e = 14.39 m
I_c = 0.008645 m^4

From Sheet 19:
σ_{fc} = 177 N/mm^2
σ_{ci} = 670 N/mm^2

From Sheet 8, ℓ_w = 18.0 m, which is 6 times ℓ_R and ℓ_R has been calculated in accordance with 9.6.4.1.1.2, so:
Take $\sigma'_{ci} = 1.25\sigma_{ci}$ = 838 N/mm^2

From Sheet 5:
d_2 = 2137 mm

$$M_y = \frac{5\times205\times10^6\times0.008645\times0.00297\times2.137}{36\times14.39(1-177/838)}\left\{1+\frac{(36/14.39)-1.25}{2.8+3.5(177/838)^2}\right\}$$

M_y = 196.0 kN

No directly applied lateral load, so $M'_y = M_y$

M'_{Dy} = (335 × 8.645 × 10^9 / 500) / (1.05 × 1.1) = 6272 kNm

$$\frac{M}{M_D}+\frac{M'_y}{M'_{Dy}} = \frac{54770}{57420}+\frac{196.0}{6272} = 0.954+0.031 = 0.985 < 1 \text{ OK}$$

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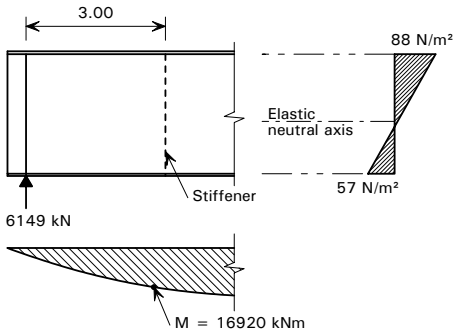
Intermediate Stiffeners

Loading

Axial loading F = F_{tw} + F_{wi} + the load transferred from interaction with the crossbeam) 9.13.3

Bending M = $(F_R + F_c)d$ + bending due to eccentricity of F

Consider the stiffener adjacent to the end frame, where F_{tw} will be greatest, and assume that the F_R and F_c forces are those determined as the greatest effects in a U-frame



To determine F_{tw} first evaluate τ_o 9.13.3.2

$$\tau_o = 3.6E\left\{1+\left(\frac{b}{a}\right)^2\right\}\left(\frac{t_w}{b}\right)^2\sqrt{1-\frac{\sigma_1}{2.9E}\left(\frac{b}{t_w}\right)^2}$$

To evaluate σ_1 calculate the stresses in the main girder at the position of the first U-frame (assume bottom doubler extends over full span, use moduli for 20 mm web).

End reaction in main girder = 6115 kN (Sheet 12)

Moment = $6115 \times 3.00 - (166.18 + 171.90) \times 3.00^2/2$ = 16820 kNm

Stress at top of web = $16820 \times 10^6 / 1.921 \times 10^8$ = 88 N/mm²

Stress at bottom of web = $16820 \times 10^6 / 2.955 \times 10^8$ = 57 N/mm² (tension)

Hence σ_b = 73 N/mm² and σ_1 = 16 N/mm²

Take τ = $6115 / (2560 \times 22)$ = 109 N/mm² (using a 22 mm web)

$$\tau_o = 3.6 \times 205000\left\{1+\left(\frac{2560}{3000}\right)^2\right\}\left(\frac{22}{2560}\right)^2\sqrt{1-\frac{16}{2.9\times 205000}\left(\frac{2560}{22}\right)^2} = 75\,\text{N/mm}^2$$

Panel length a is greater than length of stiffener ℓ_s therefore:

F_{tw} = $(\tau - \tau_o)\, t_w a$ = $(109 - 75) \times 22 \times 3000$ = 2244 kN

To determine F_{wi} (for buckling check) consider the slenderness of the effective stiffener section (see Sheet 6 for details of effective section). 9.13.3.3

$$\lambda = \frac{\ell_s}{r_{se}}\sqrt{\frac{\sigma_{ys}}{355}}$$

(Stiffener section properties with 22 mm web calculated as for section with 20 mm web, see Sheet 6)

σ_{ys} = 345 N/mm² and r_{se} = $\sqrt{5.639\times 10^8/31480}$ = 134 mm

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$$\lambda = \frac{2560}{134}\sqrt{\frac{345}{355}} = 18.8$$

From Figure 24, with $\lambda = 18.8$, $k_s = 0.036$

$$F_{wi} = \frac{\ell_s^2}{a} t_w k_s \sigma_R$$

Since there are no longitudinal stiffeners

$\sigma_R = \tau_R + \left(\sigma_1 + \frac{\sigma_b}{6}\right)$ where τ_R is the lesser of τ and τ_o

$\sigma_R = 75 + 16 + 73 / 6 = 103$ N/mm^2

$$F_{wi} = \frac{2560^2}{3000} \times 22 \times 0.036 \times 103 \times 10^{-3} = 178 \text{ kN}$$

Force transferred from the cross girder is the dead load of 1.5 m length of deck and the live load (taken to be the maximum due to RU load on most heavily loaded cross girder)

Dead load component $= 140 \times 1.5 = 210$ kN

Live load component $= 2 \times 250 \times$ DF $\times 5.07 / 9.64$

Where DF is the dynamic factor for shear, given by:

$$0.82 + \frac{1.44}{\sqrt{19.28} - 0.2} = 1.164$$

Live load component $= 500 \times 1.164 \times 0.526 = 306$ kN

Total load from cross girder $= 210 + 306 = 516$ kN

Horizontal forces causing bending of the effective stiffener.

$F_R = 66.7$ kN (acting in either direction) (Sheet 19)
$F_c = 29.0$ kN (acting outward on the most heavily loaded U-frame) (Sheet 20)

Strength of effective stiffener section

Stresses at the top of the web

Axial stress due to $F_{tw} = 2244 \times 10^3 / 31480 = 71$ N/mm^2

Bending due to eccentricity of F_{tw} from centroid of stiffener

$M = 2244 \times 118 \times 10^{-3} = 265$ kNm

Stress in web $= M / Z = 265 \times 10^6 / 4.78 \times 10^6 = 55$ N/mm^2

Stress in stiffener outstand $= M/Z = 265 \times 10^6 / 1.855 \times 10^6 = 143$ N/mm^2 (tension)

Total vertical stress in web $= 71 + 55 = 126$ N/mm^2

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CALCULATION SHEET

Job No.	BCR 876	Sheet 28 of 29		Rev
Job Title	Design guide for steel railway bridges			
Subject	Example: Design of girders for a half through plate girder bridge			
Client		Made by DCI	Date	Dec 2003
		Checked by CHP	Date	Feb 2004

Equivalent stress in web is given by: 9.13.5.1

$$\sigma_e = \sqrt{(\sigma_1 + k\sigma_b)^2 + \sigma_{es2}^2 - \sigma_{es2}(\sigma_1 + k\sigma_b) + 3\tau_R^2}$$

k is the lesser of 0.77 and $2y/b$. At the edge of the panel $2y/b = 1$ so $k = 0.77$

$$\sigma_e = \sqrt{(16 + 0.77 \times 73)^2 + 126^2 - 126(16 + 0.77 \times 73) + 3 \times 109^2} = 218 \text{ N/mm}^2$$

$$\text{Strength} = \frac{\sigma_y}{\gamma_m \gamma_{f3}} = \frac{345}{1.05 \times 1.1} = 299 \text{ N/mm}^2 \quad \text{OK}$$

Total stress in stiffener = 71 – 143 = –72 N/mm² (i.e. tension) OK

Stresses at level of the NA of the cross girder

Ignore the tensile load due to the shear transmitted from the cross girder

Vertical stresses in web and stiffener due to F_{tw} are unchanged

Bending due to horizontal restraint forces at top flange level

Ignore F_c, because it acts outwards and reduces the compressive stress in the web. Take F_R as acting inward. Hence M = 66.7 × 2137 /1000 = 143 kNm

Stress in web = 143 × 10^6 / 4.78 × 10^6 = 30 N/mm²

Stress in stiffener = 143 × 10^6 / 1.855× 10^6 = –77 N/mm² (tension)

Total stress in web = 71 +55 +30 = 156 N/mm²

k is the lesser of 0.77 and 2 × 799/2560 = 0.62 (NB σ_b is now tensile)

$$\sigma_e = \sqrt{(16 + 0.62 \times -73)^2 + 156^2 - 156(16 + 0.62 \times -73) + 3 \times 109^2} = 254 \text{ N/mm}^2 \text{ OK}$$

Total stress in stiffener = 71 – 143 – 77 = –149 N/mm² (i.e. tension) OK

Check buckling of stiffener within the middle third

Assume 2/3 of shear from cross girder is in the effective stiffener, as tensile load

Net axial load = 2244 + 178 – 2/3 × 516 = 2078 kN

Bending due to axial components = (2244 – 2/3 × 516) × 118 × 10^{-3} = 224 kNm

Bending due to F_R (taken as acting inward): M = 66.7 × 1737 /1000 = 116 kNm

Verify that:– 9.13.5.3

$$\frac{P}{A_{se}\sigma_{\ell s}} + \frac{M_{xs}}{Z_x \sigma_y} \leq \frac{1}{\gamma_m \gamma_{f3}}$$

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For single-sided stiffener, $\sigma_{\ell s}$ is given by Figure 37, Curve D, according to the value of λ.

Since σ_{ys} and σ_y are equal, the value of λ is the same as on Sheet 27, i.e. $\lambda = 18.8$ and thus $\sigma_{\ell s} = \sigma_c = 0.965 \times \sigma_{ys} = 333$ N/mm^2

$$\frac{2244\times10^3}{31480\times333}+\frac{340\times10^6}{4.78\times10^6\times345}\le\frac{1}{1.2\times1.1}$$

$0.198 + 0.206 = 0.404 \le 0.758$ OK

Other important items not covered, for reasons of brevity, include:

- Design of trimmer girders (Note that the dynamic factor on trimmers is greater than for the ordinary cross girders)
- Design of shear connectors on cross girders
- Adequacy in carrying derailed trains
- Water management
- Selection of bearings

In practice, all these items would be checked or designed.

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